-463021

MAVERICK
Construction Management Services, Inc.

June 29, 2000

Randy Sturgeon
Remedial Project Manager
USEPA Region III
Hazardous Site Cleanup Division 3HS23
1650 Arch St.
Philadelphia, PA 19103-2029

Rick Grills
Maryland Department of the Environment
2500 Broening Highway
Building 40
Baltimore, MD 21224

Re: Galaxy/Spectron Site STREAM LINER FLOAT REPORT RJS

Dear Mr. Sturgeon and Mr. Grills:

Please find enclosed Advanced Geoservices Corp's (AGC's) June 29, 2000 Evaluation Report regarding stream liner float and the downstream bypass valve. The Evaluation Report is in response to stream liner float that occurred following heavy rain on March 20 and 21, 2000.

As you are aware, startup of the ground water treatment system began on March 21, 2000. It was expected that the startup period would terminate on June 18. However, the air stripper has not yet been activated and the 100 ppb total VOCs discharge criteria has not been achieved. To obtain permission for air stripper startup and complete this phase of the project, O'Brien & Gere will submit emissions modeling information to the USEPA and MDE. The Galaxy/Spectron Group (Group) expects this information to be submitted on June 30. The Group also expects the startup phase of the project to be complete approximately two weeks after the air stripper is activated.

As you are aware, the bypass valve in the downstream cutoff wall has been open during the startup phase. In fact, the valve has remained open since April 1999 when construction of the ground water collection system and stream channel liner were completed. The bypass piping and valve were installed to drain water from the collection system while the ground water treatment system was designed and constructed. The bypass was also intended to drain ground water and prevent intolerable buoyant forces on the stream liner, should the treatment system shut down for an extended period. The valve was scheduled to be closed at the end of the startup period.

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It was apparent in 1999, that ground water did flow through the bypass. However, the flow was not monitored and specific periods during which flow occurred were not recorded. Since the liner float was observed on March 23, 2000, the bypass has been monitored. Ground water flow through the bypass was confirmed on March 28 and as late as June 13 by disconnecting the pipe within the concrete vault. Based on its evaluation of the site conditions, AGC now recommends that the bypass valve remain open beyond the startup period.

To protect the integrity of the stream liner and the effectiveness of the removal action, the Group respectfully requests that the USEPA and MDE permit the bypass valve to remain open until August 31, 2000. During this period, the Group will implement the following work to mitigate the situation.

- A flow meter will be connected to the bypass piping to quantify the ground water flow passing through the downstream cutoff wall. The flow meter is expected to be operational by July 21.

- Ground water will be preferentially pumped from Collection Manhole #3 in an attempt to lower ground water levels in the area where liner float occurred. OBG is currently pumping approximately 25 gpm of the total 50 gpm from Manhole #3. The pumping rate will be slowly increased from 25 gpm to approximately 40 gpm from Manhole #3 by the end of July.

- Ground water levels in select monitoring wells around the site will be monitored on a weekly basis. Since the liner float was observed on March 23, the Group has monitored ground water levels as part of the evaluation process.

- Large boulders will be placed in the stream channel in the area where liner float occurred. These boulders will be similar to those placed in the stream as part of the restoration effort. This additional surcharge load will resist buoyant forces acting on the liner. The boulders will be carefully placed using a crane. This work is expected to be complete by July 21.

- A site investigation will be performed to identify potential conduits that may convey surface water flow into the subsurface. Geophysical testing will be performed during this investigation. The geophysical testing will be targeted at potential conveyance structures that appear on blue prints of the former paper mill. Any conduits that are identified during this investigation will be grouted to prevent the infiltration of surface water. We expect the investigation and the grouting operation to be complete by August 4.

The Group believes that this approach is prudent. Closing the valve at this time could lead to liner float and potential damage to geomembrane seams and batten strip connections. Such damage would be extremely difficult to detect and would impact the integrity of the ground water/surface water isolation system.

MAVERICK

The effectiveness of these measures will be monitored during the month of August. The goal is to quantify, reduce and eventually eliminate ground water flow through the downstream bypass valve. If the ground water level within the collection system can be maintained below the invert elevation of the bypass pipe, then the associated buoyant force should not be large enough to lift the stream liner.

The Group will coordinate closely with the USEPA and MDE during this period. Maverick Construction Management will submit monitoring results and will keep the agencies informed on the progress of the work. In addition, the No Fishing or Swimming signs will be maintained downstream in the vicinity of the bypass discharge. If the proposed mitigation measures are not sufficient and flow continues to occur through the bypass, then the Group will have the necessary data to consider alternative options for addressing the situation.

We appreciate your consideration of this proposed plan for mitigating the liner float situation. If you have any questions, please call me at 610-783-6202 (office) or 610-659-9527 (cellular).

Sincerely,

TM Joness

Timothy M. Joness, P.E.

cc: The Galaxy/Spectron Group

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June 29, 2000

95-227-14

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Mr. Tom Morris
Manager Superfund Program
IBM
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Somers, NY 10589-0100

RE: Updated Evaluation Report
Galaxy/Spectron Site

Dear Messrs. Branchfield and Morris:

The following is an updated evaluation report prepared by Advanced GeoServices Corp (AGC) to address liner float in the Galaxy/Spectron Stream Liner System. The draft of this report was submitted in April, 2000.

INTRODUCTION

On March 21 and 22, 2000 a 5.17 inch rainfall was experienced in the vicinity of the Galaxy/Spectron Site. Because of the significance of the precipitation event (approximately a 5-year precipitation event (Type II 5-year 24 hour SCS rainfall = 4.2 inches)), AGC sent a technician to the Site on March 23, 2000. The Site visit identified a distinct area of liner float immediately above the downstream cutoff wall as shown on the attached photograph (Attachment 1). A slight bulge in Area 1 was also noted; however, liner float in this area was not confirmed by AGC.

A video taken during the March 23, 2000 visit was reviewed the following day by the AGC design engineers. AGC then reported our initial observations to the Group and initiated an evaluation at the direction of the Group. The findings, conclusions and recommendations from the evaluation are presented herein.

FINDINGS

Construction of the stream separation system was completed during the Spring of 1999. Since completion of construction and through mid-March, 2000 the system has remained in a "passive" mode with no pumping or treating of groundwater from beneath the liner. Rather, groundwater discharges through pipes from one lined section to the next until discharge occurs into the creek below the downstream cutoff wall. No liner float has been observed by AGC during that period of time, even after the 10 inches of precipitation received during Hurricane Floyd. This indicates that the system has been functioning as designed, separating surface water from groundwater and conveying groundwater to the removal/discharge point. The initial occurrence of the float observed on March 23, 2000 is unknown, but is believed to be about March 22, 2000.

OBG began start-up operation of the treatment system on March 21, 2000. From March 21 to April 4, start-up had consisted of processing 9,000 to 18,000 gallons per day from MH-1 and/or MH-3. Following April 5, full scale processing began at 72,000 gpd from all 3 manholes. Water levels within the manholes were consistently above the collection system risers during the study period and remain above the risers in MH-2 and MH-3 at the time of this report. The bypass system value has remained open throughout the 90-day start-up period.

During the late afternoon of March 23, 2000, OBG removed a section of the bypass piping (at the downstream cut off wall) between the backflow preventor and the valve, for the purpose of confirming that the bypass was not clogged. Observations made at that time where as follows:

- The valve at the downstream cutoff wall was open at the time of initial inspection (and was then closed to allow removal of the bypass piping between the backflow preventor and the valve);
- The 4" pipe from the valve had some iron precipitation but flow was unobstructed;
- The backflow preventor could not be removed because of backpressure downstream of the valve (indicating that the piping and the diffuser below the cutoff wall maintain a hydraulic connection with the creek);
- The valve was opened while the removed section was still off and full flow under pressure "shot" out of the pipe (indicating no restriction of the conveyance system above the valve); and
- Following reassembly, the valve was re-opened and iron clouds were noted in the areas of the diffusers (indicating that the diffusers were not clogged, conveying water from within the collection system to the creek).

On March 27, 2000, Paul Stratman and Todd Trotman (AGC) visited the Site to review conditions. Observations made at that time were as follows:

- Liner float was similar to the conditions documented on video the previous week;
- The valve was open at the downstream wall;

- Approximately 15 inches of water was backed up above the invert of the bypass pipe between Sections 1 and 2 (indicating a backwater condition between Sections 1 and 2);

- An artesian flow condition was observed from a pipe sticking approximately 3 feet above the surrounding grade near the former boiler house on the plant site indicating a source of water under pressure being supplied to that area. Surrounding groundwater elevations remained below the ground surface at that time indicating that the artesian condition was not related to shallow site groundwater;

- No groundwater seeps/springs were noted along the west property line;

- The gabion cover mesh in the area of float above the downstream cut off wall had popped several "bull-rings"; and

- No flow or iron staining was visible or otherwise apparent from beneath the liner along the downstream cut off wall or longitudinal wall indicating no leakage from beneath the liner.

The collection system valve in manhole MH-1 was shut and the manhole pumped below the riser. When the manhole valve was re-opened a significant amount of inflow was observed. During subsequent visits, additional information was collected as part of this evaluation. That information is summarized below.

Influent Concentration Analysis

AGC evaluated influent water quality from each sump as collected at two points in time: March 1999 OBG flow testing; and April 2000 OBG start-up testing. The purpose of the evaluation was to compare 1999 and 2000 influent quality for indications of dilution which may result in large amounts of surface water were introduced to the system. Tables containing select data and graphical presentation (by sump) are provided in Attachment 3. The following observations were made:

- Sump 1 - Concentrations of select VOCs are within the same range, but slightly lower in the 2000 influent than in the 1999 influent.

- Sump 2 - Most VOCs are within the same range for the 1999 and 2000 influent with the exception of the highest concentration VOCs (Methylene Chloride, 1,1,1-TCA) which were two to three times less concentrated in the 2000 influent.

- Sump 3 - Concentrations of the selected VOCs are within the same range in both years.

Based on the above comparison, it is AGC's opinion that the increase in flow to the sumps is not the result of a leak in the liner, which would result in significant dilution of the influent. The water that is getting into the system appears to have enough contact time with the site that its chemical quality is similar to the previously sampled system influent.

Completion of System Start-Up

As of a site visit on April 5, 2000, OBG was completing system start-up. On this date MH sump 2 was brought on line with a total of 50 gpm being extracted from all three sections. The flow rate was controlled by regulating the pumps, not the availability of the water. The accompanying hydraulic profile is beneficial in understanding the hydraulics of the systems. Calculations for system collection capacity and the capacity of the bypass system are included in Attachment 4.

- Section 1 Collection System: Since beginning full-scale treatment system operation, yield is less than 14 gpm to collection manhole MH-1 and water levels are consistently at about elevation 199.4 (above the riser elevation of 198.20). Based on a March 30, 2000 pump-down testing, the Section 1 collection piping capacity exceeds 105 gpm.

- Section 2 and 3 Collection Systems: Sections 2 and 3 are designed to operate independently under normal pumping operations. During the April 5, 2000 site visit, a static water level of 199.4 was observed indicating that the bypass piping (invert 198.7) was full and that the sections were hydraulically connected. The hydraulic connection was confirmed during a manhole pumping test as the water levels in MH-2 went down when MH-3 was pumped. A combined pumping rate for MH-2 and MH-3 on the order of 36 gpm lowers the water level below the 198.7 bypass elevation, inferring that combined yields are on the order of 35 gpm from the stream sections plus water moving through the bypass.

  The hydraulic capacity of the collection piping system for MH-2 exceeds 150 gpm based on the March 30, 2000 and the April 4, 2000 pumping studies. The capacity of the MH-3 collection piping system is greater than 300 gpm based on the April 12, 2000 evaluation.

- Bypass Piping System

  The bypass between Sections 2 and 3 is functioning well based on data from the static level study of April 5, 2000. The hydraulic connection between Sections 1 and 2 was not proven, but there is no indication suggesting the bypass (199.2) is not similarly positive.

Opening and closing the bypass cutoff valve and observations by OBG confirms that significant flow is discharged to Little Elk Creek via the diffuser system. The actual flow rate is dictated by the difference between head under the liner and the water level in the creek. Capacity of the bypass prior to any float occurring is on the order of 200 gpm. The estimated groundwater flow utilized for the design was 50 gpm.

HYDROGEOLOGIC EVALUATION

Flow Design Basis

Pre-construction estimates of groundwater flow to the stream liner collection system were comprised of two elements:

- Overburden discharge (OB)
- Bedrock discharge (BR)

Work conducted by ERM in 1994 estimated the contribution from bedrock discharge to the stream section of Spectron to be 14.1 and the overburden contribution to be 5.9 gpm. AGC used the ERM data and the extended stream channel preliminary design (AGC, November, 1996) to re-estimate the design discharge to the system. The results of this estimate were:

- Bedrock discharge estimate = 15 gpm
- Overburden estimate = 4.5 gpm

Based on observations made at the site, it is estimated that on the order of 200 gpm of groundwater are flowing through the system under current conditions. AGC focused its hydrogeologic evaluation on the bedrock and overburden conditions to determine if this excess discharge to the system could be attributable to either unit under current conditions.

Based on the observation made at the site in April 2000, elevated water levels exist in the overburden unit relative to historical levels (see Table 1 attached). AGC evaluated the following to determine if the following conditions could increase the overburden discharge to a level which would result in the observed flows:

1) increased water levels
2) increased hydraulic gradient
3) increase in hydraulic conductivity

Using the 1994 estimate of 5.9 gpm from the overburden, the following observations are noted (see Attachment 2 for calculations):

- Increasing the hydraulic gradient (i) by 2.5 increases the OB discharge to 19.5 gpm;
- Increasing the saturated thickness (b) by a factor of 2 (which would result in water levels above the ground surface) increases the OB discharge to 11.8 gpm; and
- Increasing the hydraulic conductivity (k) by a factor of 10 could result in a OB discharge of up to 68 gpm.

AGC concluded from the above evaluation that in order to produce the volume of water observed entering and flowing through the collection system, the overburden k estimates used during the design would have to have been between 1 and 2 orders of magnitude low (i.e., 10 to 100 times low). Therefore, AGC conducted the following evaluation of the site k values to determine if the estimated k values (ERM, 1994) were accurate.

Using performance data from the post collection system construction flow tests (OBG, 1999), AGC "back-calculated" k values using the following equation:

$$Q = kiA$$

where Q was known from OBG flow testing, k was left unknown, and A and i were known from site water level measurements. The resulting range of k values calculated for each stream segment range between 1.0×10^{-4} ft/sec to 3.6×10^{-7} ft/sec. These values are within the range of values calculated by ERM and used by AGC to evaluate design flows.

Finally, AGC used the 1999 calculated k values and the 2000 site groundwater conditions to estimate expected 2000 overburden contributions in sections 1, 2 and 3. These estimates resulted in the site overburden yielding approximately 12 to 15 gpm, far less than the several hundred gpm estimated to be flowing through the site collection systems in early 2000.

Based on the above, AGC has concluded that the k values used in design were still valid and the increased flow in the system cannot be due solely to higher than normal water levels and the resultant discharge of water through the overburden unit.

Similarly, water level elevations in bedrock unit (beneath the creek) are within the same range as in 1994 (see Table 1).

Angle Wells

Water levels in angle wells (AW-1, AW-2) on the east side of Little Elk Creek were found to be very close to, or lower than the historic (ERM, FRI, January 10, 1994, February 7, 1994) water levels. The angle wells are set in bedrock beneath the creek to measure the position of the bedrock watertable. This indicates conditions similar to pre-design conditions on which the design was based. Therefore, additional contribution from the bedrock unit beneath the creek is not expected.

Water levels in newly installed angle wells AW-3 and AW-3-S, located northwest of VW-1, have been measured weekly since the end of April 2000. The potentiometric surface elevations of the bedrock groundwater at this location indicate the following:

- The groundwater elevations are consistent with the surrounding potentiometric surface (i.e., water level elevations in VW-1 and MW-8); and
- There is an upward vertical gradient between the deep and shallow bedrock zone in this area (the enclosed table presents recent site water level data).

Plant Wells

Observed water levels within wells located on the plant are slightly higher with respect to the 1994 historic data (ERM, FRI) wells. The following pattern is observed:

1. Along the stream bank north of the treatment building (MW-9) no rise in GWL is observed. Water level elevation in MW-8 are approximately 6" higher in 2000 than in 1999 (see Table 1). 0.5' = 6"
2. The site area south of the OBG treatment building and encompassing MH-3 and MW-11, and to some extent, VW-3 has GWL's 0.5' above the historic 1994 data. Portions of the area are underlain by pockets of porous masonry rubble fill (water levels in the overburden in this area during April, May and June 2000 were as much as 7 feet higher than levels observed during March 1999).
3. The western section of the plant site is situated at the base of the regional valley side slopes where topographic features are believed to account for the GWL's in VW-1 and MW-8 being 0.9 to 1.6 ft. above their 1994 and 1999 historic averages, respectively.

Based on these observations, the GWL's beneath the plant are slightly elevated with respect to 1994 data. The 0.9 to 1.6 ft of increased head in the western plant sector is probably most indicative of the seasonal groundwater rise. The 6 inches of rise in the areas north of the treatment building is also attributed to seasonal groundwater and an increase in head created by the collection system operating in passive mode. The greater than 3 ft. higher groundwater levels in the area south of the treatment area is uncharacteristic relative to historic data and other observed groundwater conditions at the Site (i.e., 1999 water levels), especially knowing that a good hydraulic connection exists between MW-11 and the collection system. Therefore, it appears that an additional input of water exists in this area, possibly short circuiting from above the dam and through the site.

PRELIMINARY CONCLUSIONS

AGC has the following preliminary conclusions regarding the cause of the liner float and the affect of the liner float on the integrity of the existing liner system.

Cause of Liner Float

Groundwater levels in the vicinity of MW-11 were holding at a uniform elevation of 204 feet MSL (±) and are now declining slowly to approximately 202 feet MSL. This represents a groundwater mound that is higher than the nearby overburden monitoring wells (i.e., MW-12) along the creek; higher than historic monitoring well levels available since 1994; higher than the observed elevations within the collection system sumps; and higher than the water surface has been in Little Elk Creek since hurricane Floyd. The isolated mounding suggests that a source is conveying water directly to the area of MW-11 while maintaining a head greater than areas upstream. LOCATION wells

The direct introduction of water into the area of MW-11 is occurring at an estimated rate of approximately 300 gallons per minute. As a result of the very significant flow, the capacity of the groundwater collection and bypass system installed as part of the stream lining system is exceeded (nominal capacity of 190 gpm) and subsequently experiences an increase in head beneath the liner system which cannot be rapidly relieved and which exceeds the combined load of the gabion system and the creek water level causing liner float.

It appears that the most likely mechanism of transferring water directly into the area is one or more of the subsurface pipes or chases that supported the original paper mill operations at the site. A 1947 blue print (Attachment 5) shows a complex network of underground piping, including an 8-inch diameter water line which runs the length of the site, directly into the vicinity of MW-11. That same waterline is suspected to terminate in an area mapped by ground penetrating radar as being rubble fill near MW-11. During the Pre-Design Investigation (1996), a geophysical survey was performed in an effort to locate buried utilities or structures on the southern portion of the Site which may act as artificial conduits which short circuit groundwater flow. The subsurface utility survey was conducted on the southeastern portion of the Site (from the footbridge to the area approximately 50 feet south of Providence Road Bridge) in an attempt to identify metallic and non-metallic utilities and other structures within this area. A combination of ground penetrating radar (GPR), electromagnetic (EM) and metal detection (MD) methods were used. The full report was submitted with the November 1996 Pre-Design Investigation Summary Report. The area surveyed contained many features which were interpreted as subsurface metallic utilities and several areas of high density, discontinuous reflections which were interpreted as construction or demolition debris.

One linear feature of particular interest was encountered running nearly parallel to Northing 110. This target was traced from the "Area H" building toward the eastern edge of the Site (near the creek at MW-11) at which point the feature was no longer discernible. This was the only feature identified at that time near Providence Road Bridge which may act as a potentially significant conduit groundwater on this portion of the Site.

No information is available that indicates that the hydraulic connection existed during the Focused Remedial Investigation, the pre-design investigation activities, or during construction.

Integrity of Liner System

AGC's conclusion concerning liner system integrity is that the liner system in the vicinity of the float remains intact. This conclusion is based upon field observations including the understanding that the excess head would be relieved if a rip or tear had occurred in the system; and that the odor and/or iron staining indicative of the groundwater mixing with the surface water is absent along the longitudinal and downstream walls. In addition, conservative calculations (which ignore excess slack or elongation) indicate that the tensile stress developed in the sheet of the geomembrane due to the float is 18 pounds per inch (ppi), well below the tensile strength observed during conformance testing of 100 ppi and the average destructive test weld shear strength of 89 ppi (the elongation at break during conformance testing was 20%). The strength of the batten bar (i.e., bolts secured into the concrete), exceeds the strength of the liner. Therefore, a failure in the liner, if it was to occur, would occur as a rip in the liner at the bolted connection, not a pullout of the bolt.

RECOMMENDATIONS

The presence of the unknown water source to the collection system means that water will continue to flow through the bypass during pump and treatment operations. To identify and terminate the water source, AGC makes the following recommendations:

- Provide additional ballast on the liner system immediately adjacent to the downstream cutoff wall;

- Maintain the bypass system with the valve open to allow automatic activation whenever necessary; and

- Evaluate the north end of the site to identify potential conduits that could still be conveying surface water into the subsurface.

If you have any questions about the information presented, please call.

Sincerely,

ADVANCED GEOSERVICES CORP.

Todd D. [illegible] Fr

Paul G. Stratman, P.E.
Project Manager

Wm. K. Richardson, Jr.

William K. Richardson, Jr.
Principal

WKR:PGS:vm

Enclosures

cc: Larry Adrian
John Alonzo
Mark Attaway
Carl Everett
Joe Keller
Peter Ramaley
Sathya Yalvigi
Tim Joness
John Cutrone

TABLE

TABLE 1
Average Water Elevations
Spectron Site

Well ID	1994	1996	1999	2000	1999 vs. 2000	1994 vs. 2000
MW-8	212.38	NT	211.43	213.04	+ 1.61 ft.	+ .66 ft.
MW-9	204.64	NT	203.41	204.02	+ 0.61 ft.	- 0.62 ft.
MW-10	201.81	202.79	202.77	NA	NA	NA
MW-11	202.53	202.46	197.10	203.99	+ 6.89 ft.	+ 1.46 ft.
MW-12	201.96	NT	199.84	201.48	+ 1.64 ft.	- 0.48 ft.
AW-1	204.25	NT	NT	202.33	NA	-1.92
AW-2	203.80	NT	NT	202.83	NA	-0.97
VW-1	209.48	NT	NT	210.37	NA	0.89
VW-3	204.83	NT	202.49	205.42	+ 2.93 ft.	+ .59 FT.

Notes:

NT - No measurement taken

NA - Not available

AR301707

WLcomparisons4-00.xls

ATTACHMENT 1

PHOTOGRAPH OF FLOAT


3/24/2000

ATTACHMENT 2

HYDROGEOLOGIC CALCULATIONS

ADVANCED GEOSERVICES CORP.

"Engineering for the Environment"™

PROBLEM: CALCULATE HYDRAULIC CONDUCTIVITY (K) OF OVER BURDEN USING 1999 SITE CONDITION. USE TO VALIDATE DESIGN K VALUES

① SATURATED THICKNESS AT OB WELLS

WELL #	OB DEPTH	1994 ① DTW	1999 ② DTW	2000 ③ DTW	Δ
MW-8	8'	4.55'	4.97'	3.62'	
SATURATED THICKNESS →		3.95'	3.03'	4.38'	1'
MW-9	11'	9.51'	9.70'	9.42'	
SAT. THICKNESS →		1.49'	1.30'	1.58'	NONE
MW-10	14'	7.89	7.18	—	
SAT. THICKNESS →		6.11	6.82	—	—
MW-11	10.5	5.37	9.76	2.41	
SAT. THICKNESS →		5.13	0.74	8.09	3'
MW-12	15	8.72	10.22	8.90	
SAT. THICKNESS →		6.28	4.78	6.10	NONE
	AVERAGE =	$\boxed{4.49}$	$\boxed{3.33}$	$\boxed{5.40}$	

SECTION LENGTH: 1 = 240' 2 = 270' 3 = 320'

② SECTION 1: CALCULATE K @ Q OF 12 GPM (036, 1999)

$Q = kiA$ $A = 240' \times 3.33' = 800' \ FT^2$

$i = 0.04 \ FT/FT$

$k = \frac{Q}{iA}$ $Q = 12 \ GPM$

$$K = 12 \ GPM \cdot \frac{1 \ FT^3}{7.48 \ g/FT^3} \times \frac{1}{iA} = \frac{1.604 \ FT^3/MIN}{0.04 \ FT/FT \cdot 800 \ FT^2}$$

$$= 0.050 \ FT/MIN.$$

$$= 8.3 \times 10^{-4} \ FT/SEC \ (\text{Composite})$$

DIVIDE BY 3 TO GET OVER BURDEN K

$$8.3 \times 10^{-4} \div 3 =$$

$$= 2.8 \times 10^{-4} \ FT/SEC$$

① 2/7/94
② 3/17/99
③ 4/7/00

SHEET 1 OF 2 PROJECT NO. 95-227 PROJECT NAME SPECTRON
BY WER DATE 4/23/00 DESCRIPTION HYDRO CALCS
CHK. BY ______ DATE 5/30/00

SECTION 2 CALCULATE K AT Q = 8 GPM (OBG 1999)

$Q = kiA$ $A: 270' \times 5' = 1350 \text{ FT}^2$
$k = \frac{Q}{iA}$ $i: 0.04 \text{ FT/FT}$
$Q: 8 \text{ GPM}$

$k = 8 \text{ GPM} \cdot \frac{1 \text{ FT}^3}{7.48 \text{ FT}^3/\text{gal}} \times \frac{1}{iA} = \frac{1.07 \text{ FT}^3/\text{MIN}}{0.04 \text{ FT/FT} \cdot 1350}$

$k = 1.98 \times 10^{-2} \text{ FT/MIN}$

$= 3.30 \times 10^{-4} \text{ FT/SEC}$

DIVIDE BY 3 TO GET OVERBURDEN K

$3.30 \times 10^{-4} \div 3$

$= 1.1 \times 10^{-4} \text{ FT/SEC}$

SECTION 3 - CALCULATE K AT Q = 15 GPM (OBG, 1999)

$Q = kiA$ $A: 320 \left(\frac{1+5'}{2}\right)$
$k = \frac{Q}{iA}$ $= 800 \text{ FT}^2$ [USE MW-11, MW-12 SAT. THICKNESS]

$k = 15 \text{ GPM} \cdot \frac{1 \text{ FT}^3}{7.48 \text{ FT}^3/\text{gal}} \times \frac{1}{iA} = \frac{2.005 \text{ FT}^3/\text{MIN}}{0.03 \cdot 800'}$

$= 8.35 \times 10^{-2} \text{ FT/MIN}$

$= 1.39 \times 10^{-3} \text{ FT/SEC}$

DIVIDE BY 3 TO GET OVER BURDEN K

$k = 4.64 \times 10^{-4} \text{ FT/SEC}$

ERM 1994 RANGE OF K VALUES IS:
$1.0 \times 10^{-4} \text{ FT/SEC}$ TO $3.6 \times 10^{-7} \text{ FT/SEC}$
AVERAGE = $4.08 \times 10^{-5} \text{ FT/SEC}$

SHEET 2 OF 2	PROJECT NO. 95-227	PROJECT NAME SPECTRON
BY WKR	DATE 4/23/00	DESCRIPTION HYDRO CALCS
CHK. BY	DATE 5/30/00	

USING 99 K VALUES CALCULATED BY USING OCC FLOW & MW - WATER LEVELS - CALCULATE Q USING 2000 SATURATED THICKNESS & GRADIENT AT SECTION 1, 2 & 3

4/2000

SECTION 3

SAT THKNS = 8' @ MW-11

$Q = K i A$

$A = 8' \times 320' = 2560 \, FT^2$

$i = 0.0095$ (4/05/00 W.L. INFO)

$k = 8.35 \times 10^{-2}$ FT/MIN (.99 CALCULATED VALUE)

$Q = 8.35 \times 10^{-2} \, FT/MIN \times 0.0095 \times 2560 \, FT^2$

$Q = 2.030 \, FT^3/MIN \quad \times 7.486/FT^3$

$Q = \boxed{15.2 \text{ GPM}}$ FROM ZONE 3 O.B.

∴ UNDER CURRENT CONDITIONS
ie - MAXIMUM K
MAXIMUM SAT. THICKNESS
EXISTING GRADIENT
ONLY 15.2 GPM CAN BE ACCOUNTED FOR FROM THE OVER BURDEN UNIT IN SECTION 3

SECTION 2

SAT THCKNS @ MW-12 = 6.1'

$Q = 0.0198 \, FT/MIN \cdot 0.03 (2000) \cdot (6.1' \times 270')$

$Q = \text{"} \cdot \text{"} \cdot 1647$

$Q = 0.978 \, FT^3/M \quad \times 7.486/FT^3$

$Q = 7.31$ GPM

SECTION 1

SAT THCKNS @ MW-9 = 1.58' & MW-10 6.82' AV = 4'

$Q = 0.05 \, FT/MIN \cdot 0.05 \cdot (240' \times 4')$

$Q = 2.4 \, FT^3/MIN$

$Q = 17.95$ GPM

SHEET 1 OF ___ PROJECT NO. 95-227 PROJECT NAME SPECTRON
BY WKR DATE 4/23/00 DESCRIPTION FLOAT SUPPORT ANALYSIS
CHK. BY ___ DATE ___

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Nov. 22, 1996 " Removal Action Pre-Design Investigation Summary Report" PAGE 22

Discharge rate used in the piezometer and bedrock flux calculations is the rate measured and documented in the FRI REPORT -

ERM

- These calculations are in appendix M. of the FRI and total 20 GPM from overburden and bedrock

- NOV 22, 1996 Report (AGC) USED Discharge value of 17 to 24 GPM from both bedrock and overburden from both sides of stream. (P. 23)

NOTES FOR 4/2000 Hydro calcs @ SPECTRON -

WKR

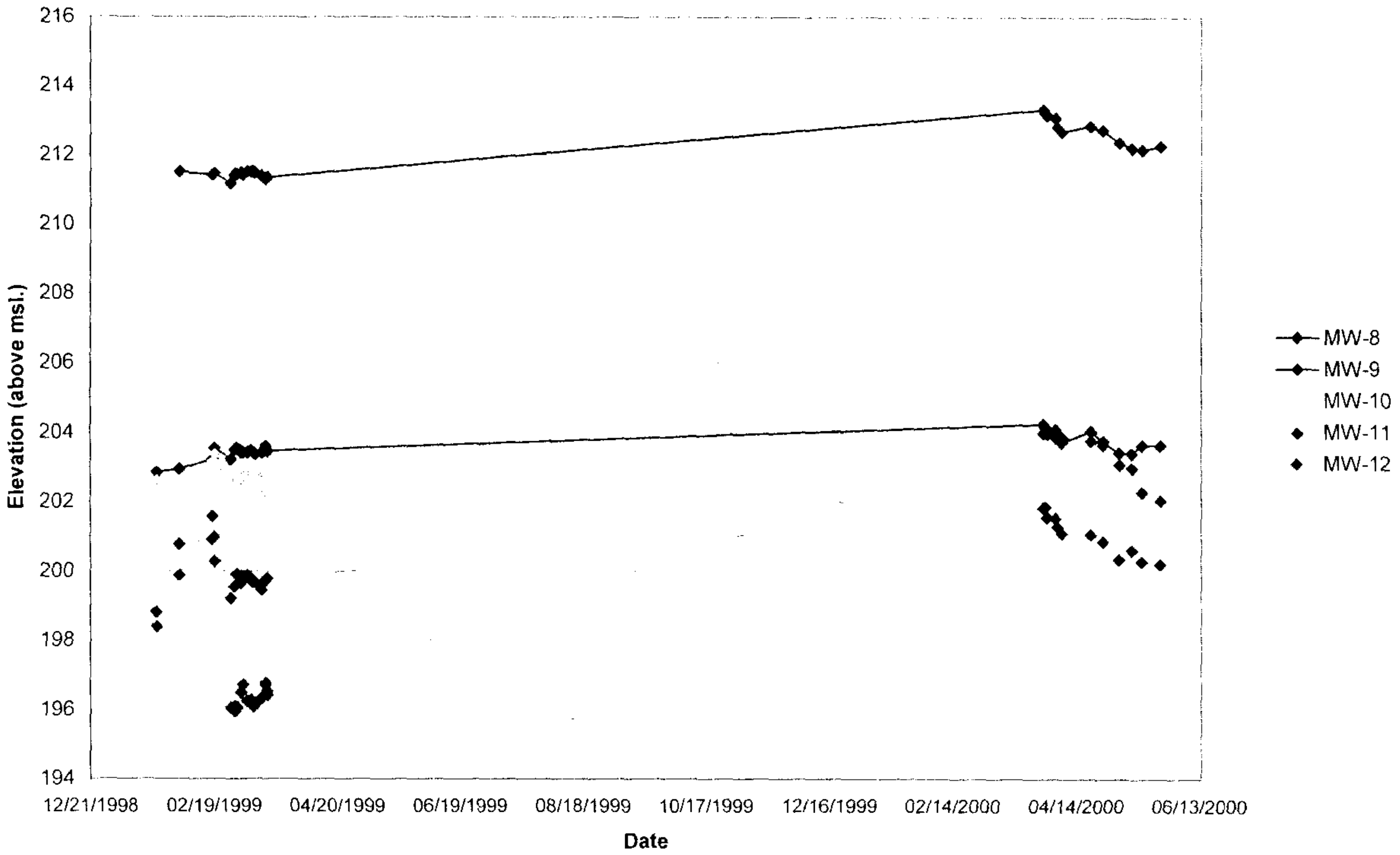
SHEET 2 OF 2 PROJECT NO. ______ PROJECT NAME NOTES

BY ______ DATE ______ DESCRIPTION ______

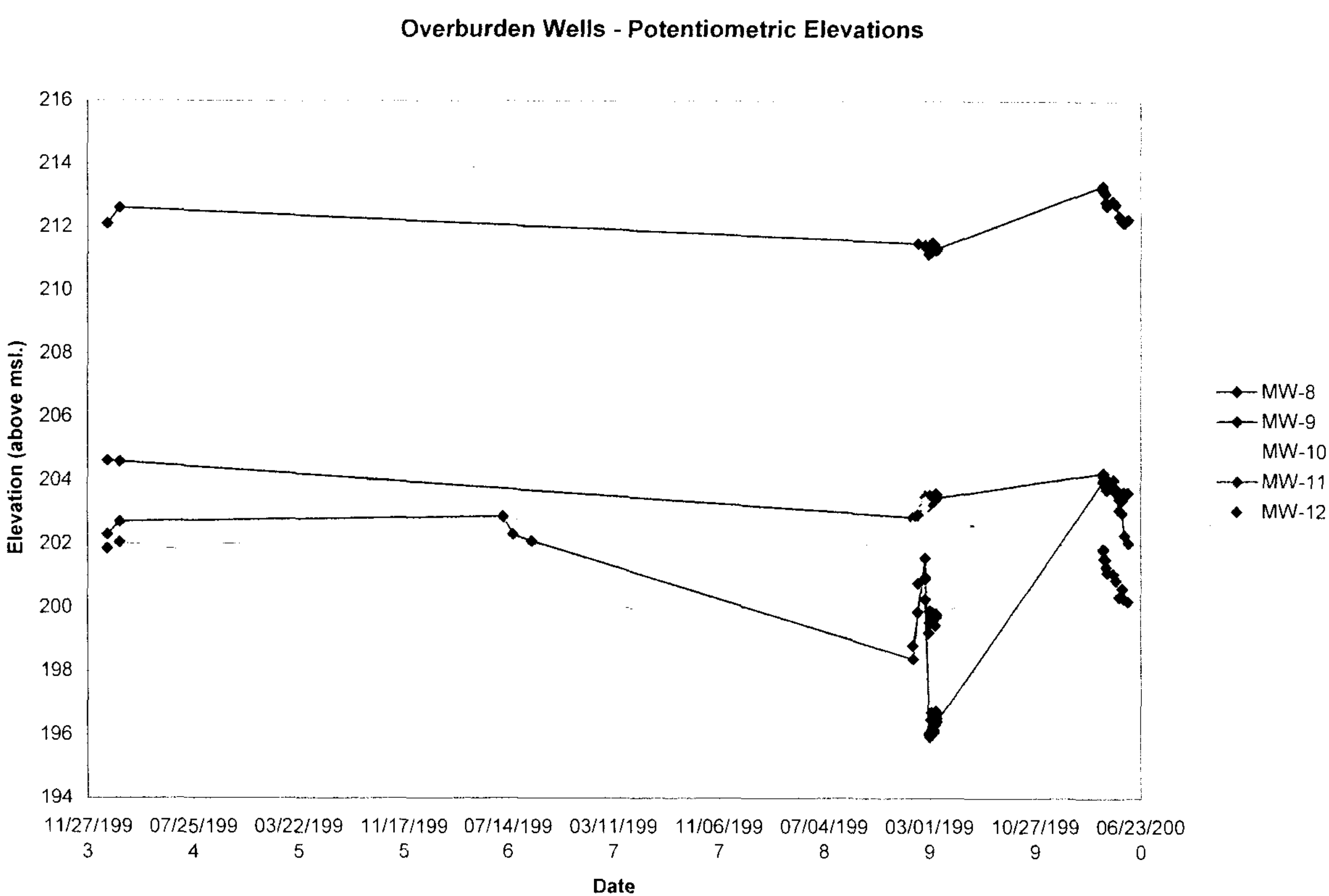
CHK. BY ______ DATE ______

AR301714

Overburden Wells Potentiometric Elevation 1/99 - 5/25



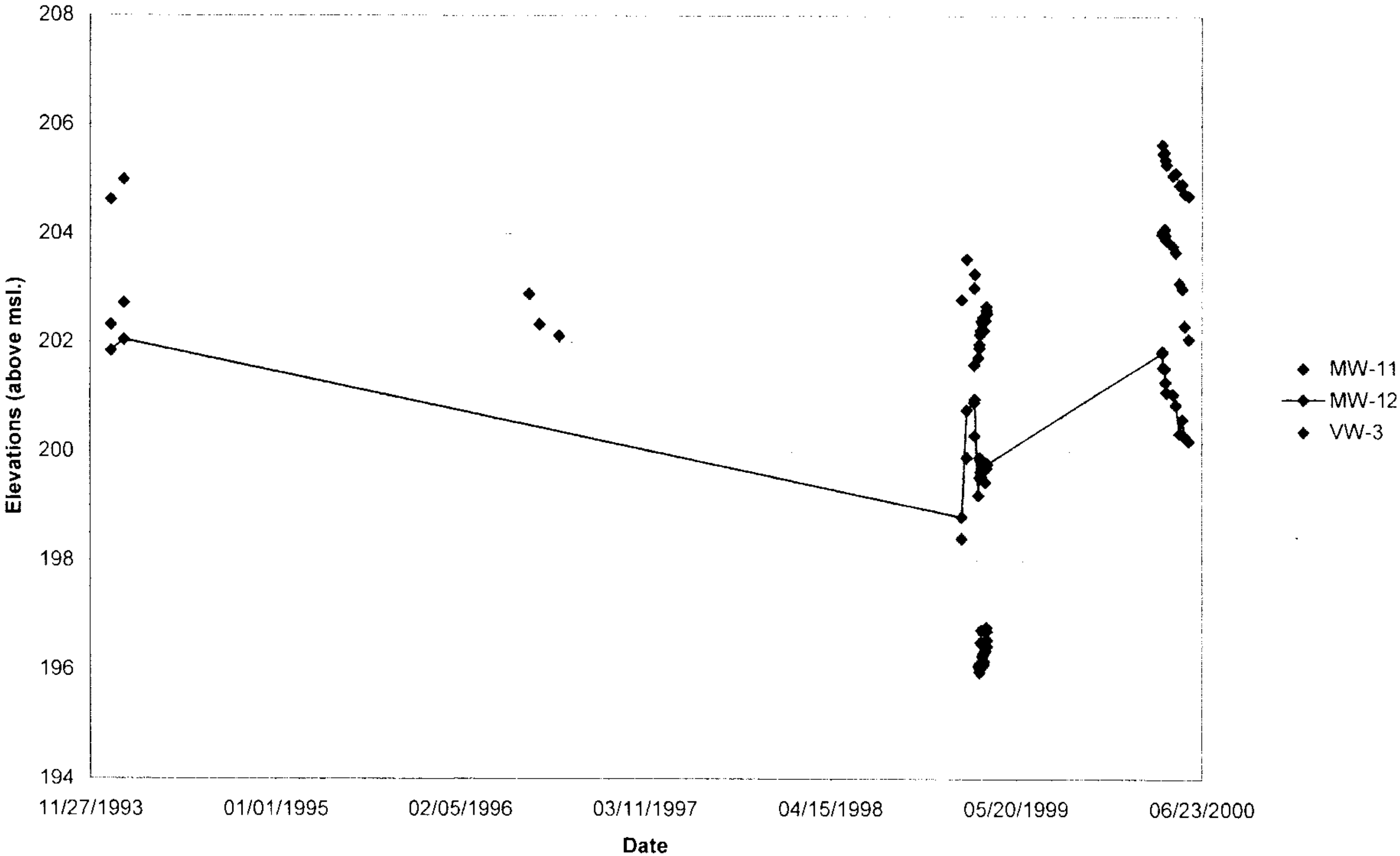
WLcomparisons4-00.xls



WLcomparisons4-00.xls

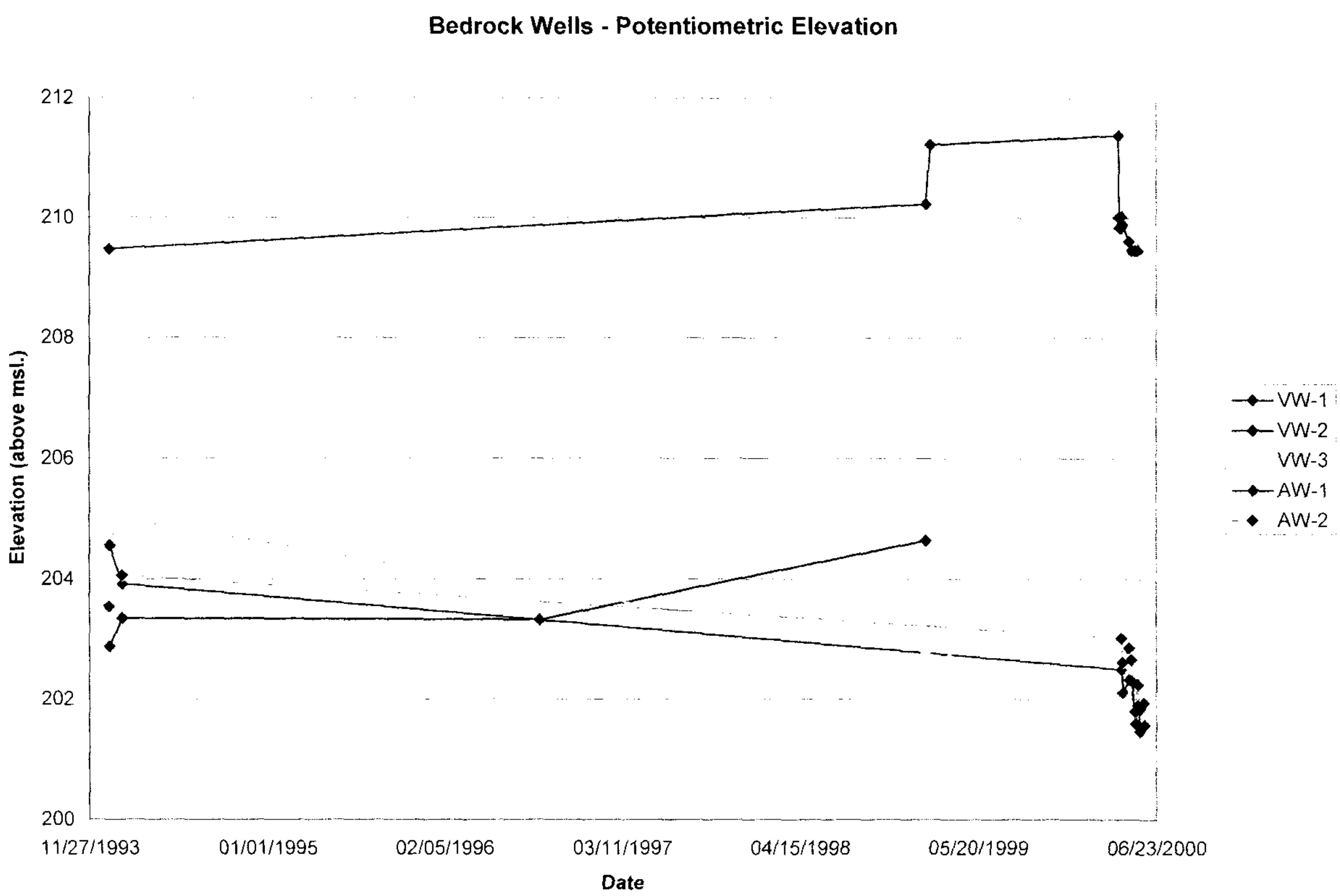
AR301716

MW-11, MW-12 and VW-3 - Potentiometric Elevations



WLcomparisons4-00.xls

AR301717



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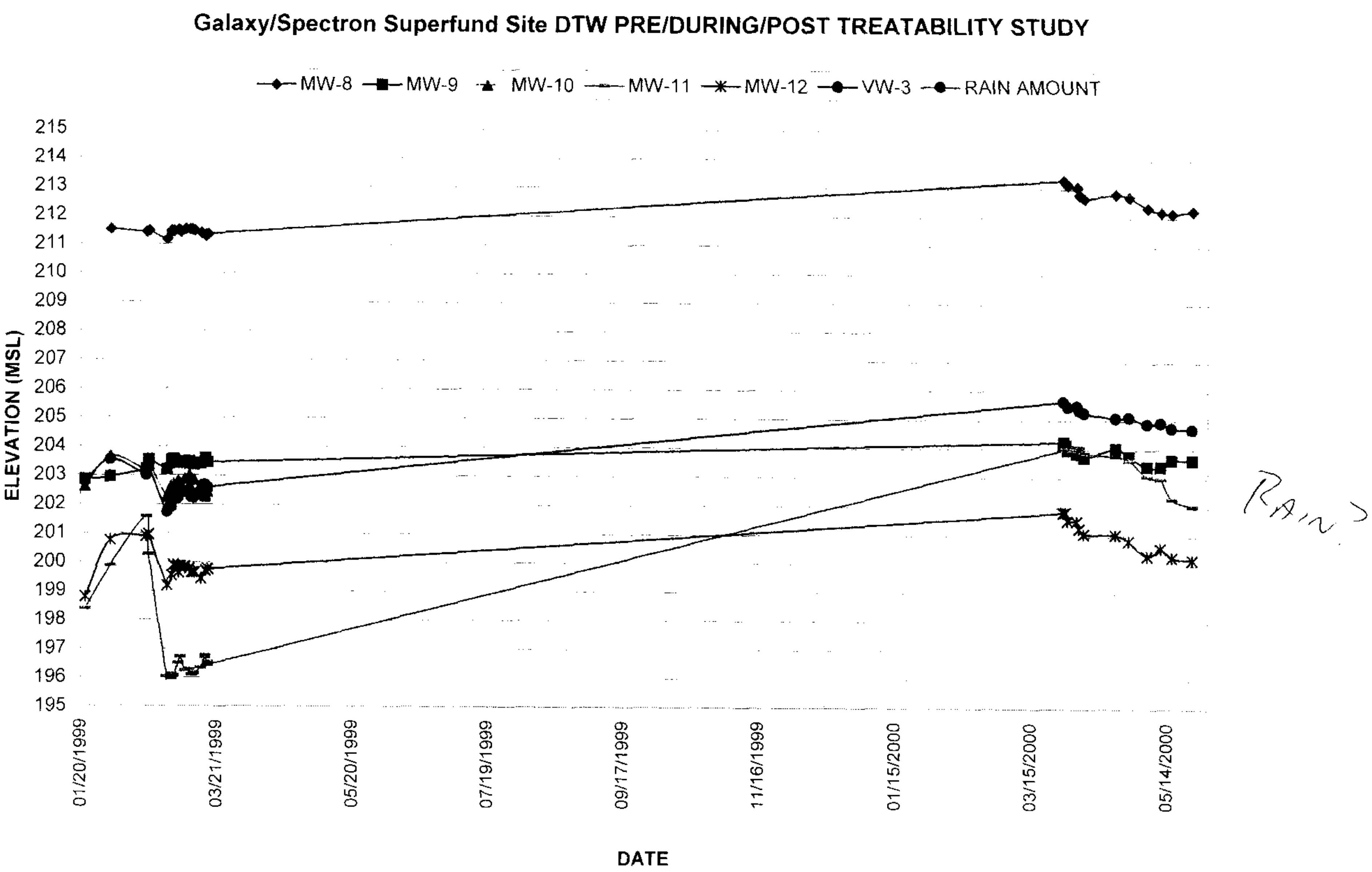
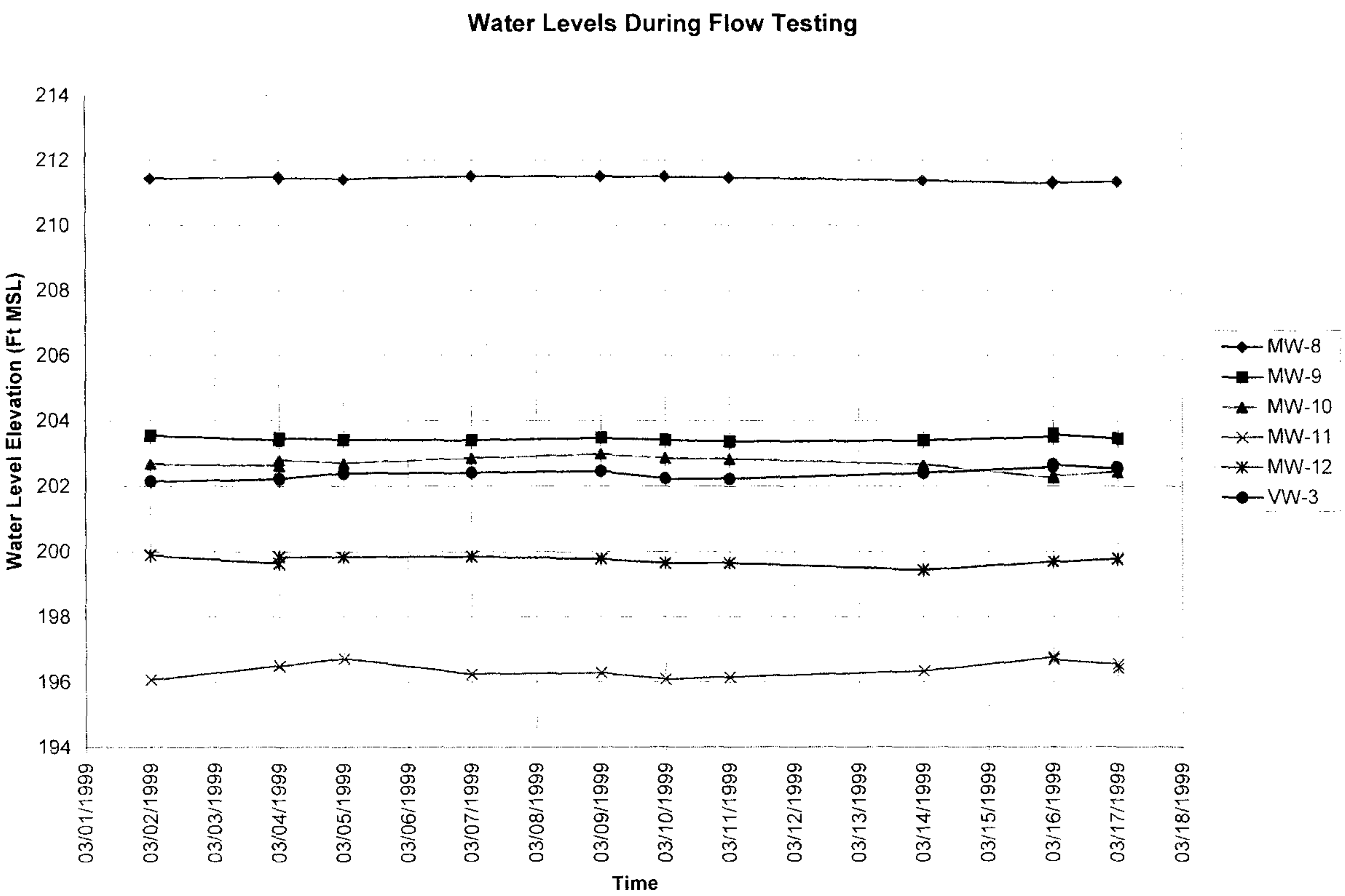
AR301718



Galaxy/Spectron Superfund Site DTW PRE/DURING/POST TREATABILITY STUDY
MW-8
MW-9
MW-10
MW-11
MW-12
VW-3
RAIN AMOUNT
ELEVATION (MSL)
215
214
213
212
211
210
209
208
207
206
205
204
203
202
201
200
199
198
197
196
195
01/20/1999
03/21/1999
05/20/1999
07/19/1999
09/17/1999
11/16/1999
01/15/2000
03/15/2000
05/14/2000
DATE
RAIN?



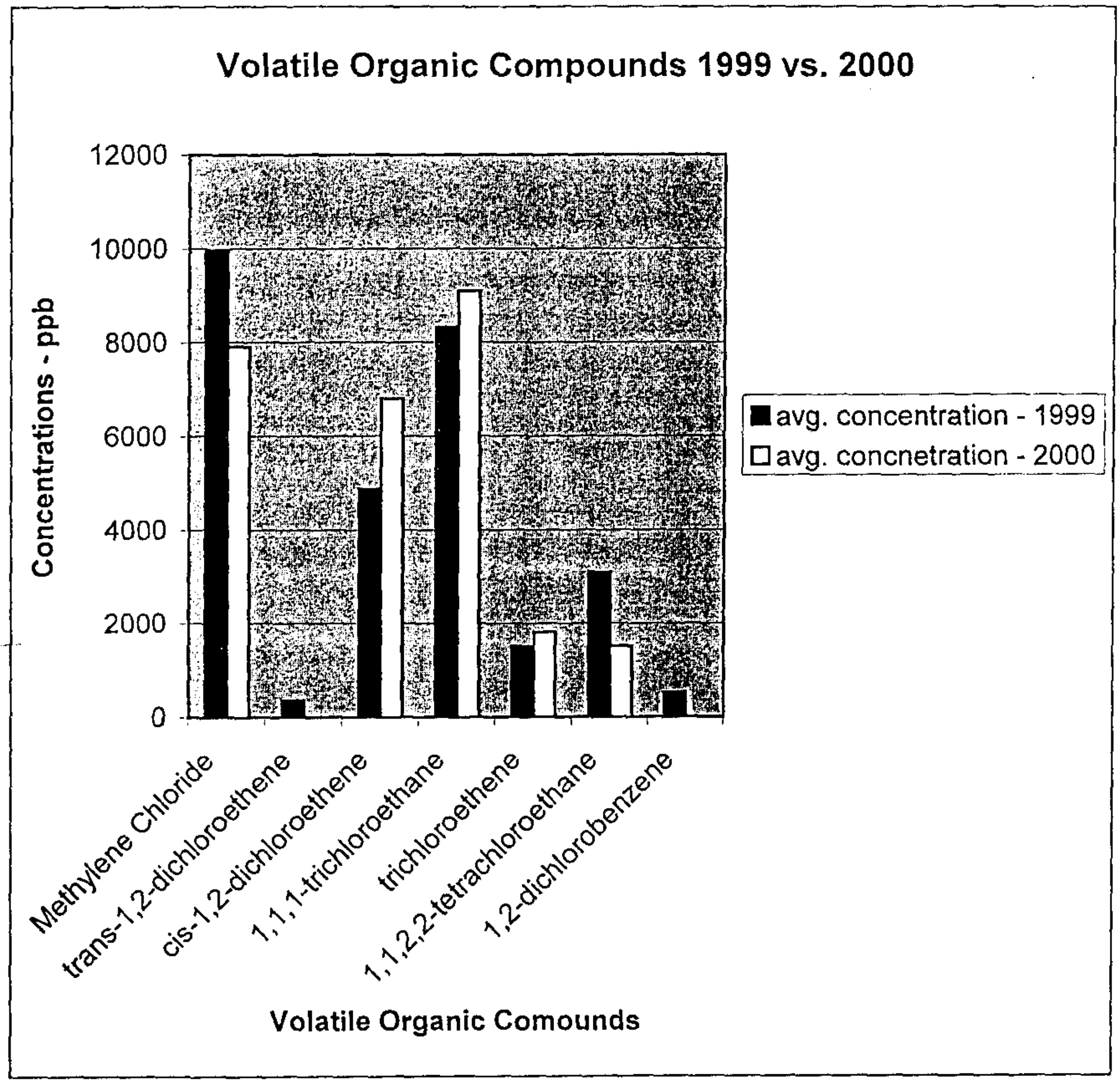
Water Levels During Flow Testing
Water Level Elevation (Ft MSL)
214
212
210
208
206
204
202
200
198
196
194
03/01/1999
03/02/1999
03/03/1999
03/04/1999
03/05/1999
03/06/1999
03/07/1999
03/08/1999
03/09/1999
03/10/1999
03/11/1999
03/12/1999
03/13/1999
03/14/1999
03/15/1999
03/16/1999
03/17/1999
03/18/1999
Time
MW-8
MW-9
MW-10
MW-11
MW-12
VW-3

ATTACHMENT 3

INFLUENT CONCENTRATIONS

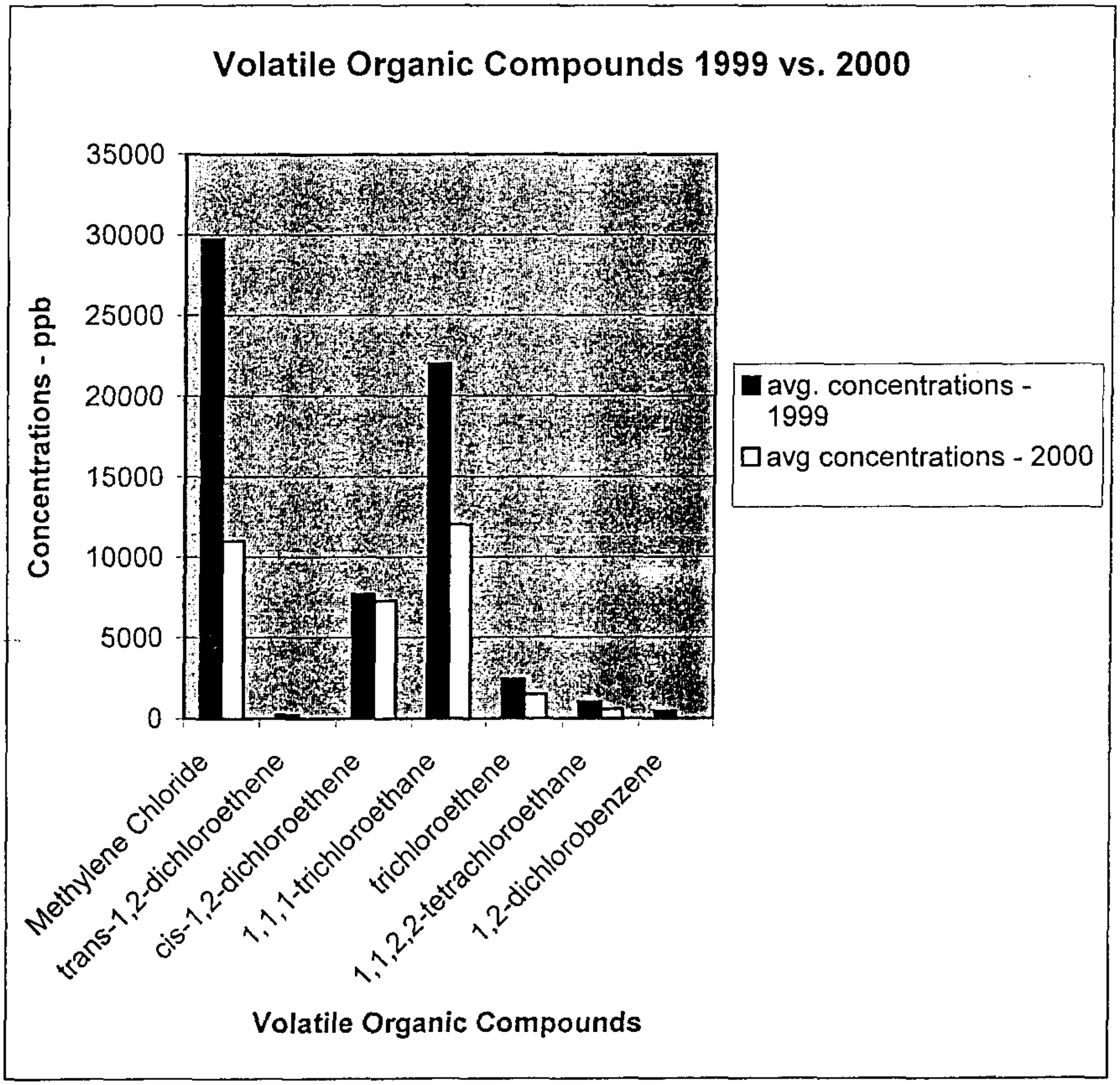
Volatile Organic Compounds
Sump #1

Volatile Organic Compound	Average Concentration - ppb (1999)	Average Concentration - ppb (2000)
Methylene Chloride	9975	7900
trans-1,2-dichloroethene	357.5	<500
cis-1,2-dichloroethene	4875	6800
1,1,1-trichloroethane	8325	9100
trichloroethene	1500	1800
1,1,2,2-tetrachloroethane	3100	1500
1,2-dichlorobenzene	522.5	<500



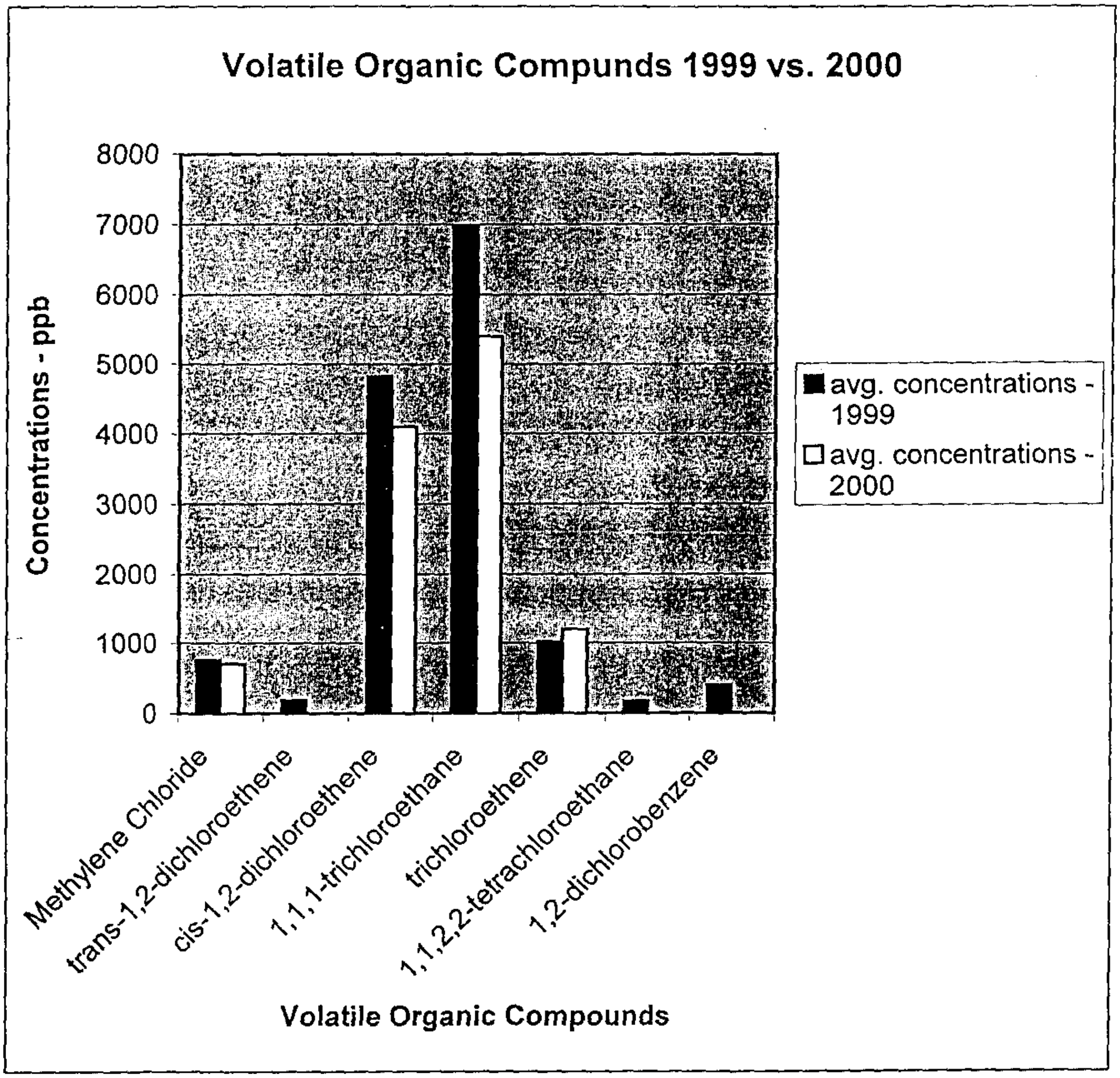
Volatile Organic Compounds
Sump #2

Volatile Organic Compound	Average Concentration - ppb (1999)	Average Concentration - ppb (2000)
Methylene Chloride	29750	11000
trans-1,2-dichloroethene	250	<500
cis-1,2-dichloroethene	7725	7300
1,1,1-trichloroethane	22000	12000
trichloroethene	2425	1500
1,1,2,2-tetrachloroethane	1030	580
1,2-dichlorobenzene	455	<500



Volatile Organic Compounds
Sump #3

Volatile Organic Compound	Average Concentration - ppb (1999)	Average Concentration - ppb (2000)
Methylene Chloride	757.5	700
trans-1,2-dichloroethene	187.5	<500
cis-1,2-dichloroethene	4825	4100
1,1,1-trichloroethane	6975	5400
trichloroethene	1015	1200
1,1,2,2-tetrachloroethane	170	<500
1,2-dichlorobenzene	400	<500



Volatile Organic Compounds

Sump #1		
Volatile Organic Compound	Average Concentration - ppb (1999)	Average Concentration - ppb (2000)
Methylene Chloride	9975	7900
trans-1,2-dichloroethene	357.5	<500
cis-1,2-dichloroethene	4875	6800
1,1,1-trichloroethane	8325	9100
trichloroethene	1500	1800
1,1,2,2-tetrachloroethane	3100	1500
1,2-dichlorobenzene	522.5	<500

Sump #2		
Volatile Organic Compound	Average Concentration - ppb (1999)	Average Concentration - ppb (2000)
Methylene Chloride	29750	11000
trans-1,2-dichloroethene	250	<500
cis-1,2-dichloroethene	7725	7300
1,1,1-trichloroethane	22000	12000
trichloroethene	2425	1500
1,1,2,2-tetrachloroethane	1030	580
1,2-dichlorobenzene	455	<500

Sump #3		
Volatile Organic Compound	Average Concentration - ppb (1999)	Average Concentration - ppb (2000)
Methylene Chloride	757.5	700
trans-1,2-dichloroethene	187.5	<500
cis-1,2-dichloroethene	4825	4100
1,1,1-*trichloroethane*	6975	5400
trichloroethene	1015	1200
1,1,2,2-tetrachloroethane	170	<500
1,2-dichlorobenzene	400	<500

ATTACHMENT 4

HYDRAULIC CALCULATION

PIPE CAPACITY CALCULATIONS

PURPOSE

The following calculations determine the pipe capacity for each of the three collection zones for varying head conditions (i.e., varying groundwater levels).

METHOD

The Bernoulli equation will be used to calculate pipe flow. The equation will be written between a point at the top of the groundwater level beneath liner and the top of the rise in the manhole. The Manning equation will be used to determine friction loss. Minor losses will be calculated based on velocity head.

ASSUMPTIONS

The following assumptions were made:

- The presence of the liner was ignored (i.e., the surface of the groundwater was assumed to open to the atmosphere - velocity head and pressure head below the liner is assumed to be zero). Since pipe capacity is being determine, the reaction of the water surface and liner was ignored. Effects of the groundwater level on the liner will be evaluated separately.

- It is assumed that the operating water level (maximum pumping level) in the sumps is maintained at an elevation below the riser pipe. Therefore, the pressure head at the sump is zero.

- The collection system in each zone has parallel piping connecting to a single collection lateral leading to the manhole. All pipes are 6 inches in diameter. Therefore, the collection capacity of the system will be determine by analyzing the longest single pipe route to the manhole.

- All soil and pipe is completely submerged in water and the inflow capacity of the pipe perforations far exceeds the pipe carrying capacity.

- The pipe is smooth HDPE. A Manning n of 0.012 was assumed.

CALCULATION SUMMARY

$$z_L + \frac{P_L}{\gamma} + \frac{U_L^2}{2g} - h_{loss} = z_s + \frac{P_s}{\gamma} + \frac{U_s^2}{2g}$$

$$z_L - h_{loss} = z_s + \frac{U_s^2}{2g}$$

Head Losses

- Friction loss

$$h_f = \frac{n^2 L U^2}{2.21 R^{2/3}}$$

$n = 0.012$

$$R = \frac{A}{P_w} = \frac{\pi r^2}{2\pi r} = \frac{r}{2} = \frac{0.25'}{2} = 0.125$$

$$h_f = \frac{(0.012)^2 L U^2}{2.21 (0.125)^{2/3}} = 2.61 \times 10^{-4} L U^2$$

- Minor Losses

$$90^\circ \text{ Elbows} = 1.10 \frac{U^2}{2g}$$

$$\text{Riser Exit} = \frac{U^2}{2g}$$

$$Z_L - \left[2.61 \times 10^{-4} L V^2 + (\#\text{ Bends})\left(1.18 \frac{V^2}{2g}\right) + \frac{V^2}{2g}\right] = Z_S + \frac{V^2}{2g}$$

ZONE 1 CALCULATION

- 2 90° Bends
- L = 244 ft
- Z_L (To be varied)
- Z_S = 197.89

$$Z_L - \left[(2.6 \times 10^{-4})(244) V^2 + (2)\left(1.18 \frac{V^2}{2g}\right) + \frac{V^2}{2g}\right] = 197.89 + \frac{V^2}{2g}$$

$$Z_L - \left[0.06344\, V^2 + 2.36 \frac{V^2}{2g} + \frac{V^2}{2g}\right] = 197.89 + \frac{V^2}{2g}$$

$$Z_L - 0.11561\, V^2 = 197.89 + 0.015528\, V^2$$

$$Z_L - 197.89 = 0.13114\, V^2$$

$$V_1 = \left(\frac{Z_L - 197.89}{0.13114}\right)^{1/2}$$

Zone 2 Calculation:

- (2) 90° Bends
- $L = 268$ FT.
- $Z_L =$ (To be varied)
- $Z_S = 196.83$ FT.

$$Z_L - \left[(2.6\times10^{-4})(268)V^2 + (2)\left(1.18\frac{V^2}{2g}\right) + \frac{V^2}{2g}\right]$$

$$= 196.83 + \frac{V^2}{2g}$$

$$Z_L - \left[.06968\,V^2 + 2.36\underset{(.0366)}{\frac{V^2}{2g}} + \frac{V^2}{2g}\right] = 196.83 + \frac{V^2}{2g}$$

$$Z_L - .1219\,V^2 = 196.83 + .01553\,V^2$$

$$V_2 = \left(\frac{Z_L - 196.83}{0.13743}\right)^{1/2}$$

AR301730

Zone 3 Calculation:

- (2) 90° Bends
- $L = 183$ FT.
- $Z_L =$ (To be varied)
- $Z_S = 195.34$ FT.

$$Z_L - \left[(2.6\times10^{-4})(183)V^2 + (2)\left(1.18\frac{V^2}{2g}\right) + \frac{V^2}{2g}\right] = 195.34 + \frac{V^2}{2g}$$

$$Z_L - \left[.04758\,V^2 + \underbrace{2.36\frac{V^2}{2g}}_{(.0366)} + \frac{V^2}{2g}\right] = 195.34 + \frac{V^2}{2g}$$

$$Z_L - .09971\,V^2 = 195.34 + .01553\,V^2$$

$$V_3 = \left(\frac{Z_L - 195.34}{0.11524}\right)^{1/2}$$

AR301731

CAPACITY CALCULATIONS
COLLECTION PIPING

Zone 1 Capacity

Groundwater Elevation (ft.)	Velocity (ft/s)	Discharge (cfs)	Discharge (gpm)	Upward Head on Liner (ft.)
197.89	0.00	0.00	0	0.89
198.00	0.92	0.18	81	1
198.20	1.54	0.30	136	1.2
198.40	1.97	0.39	174	1.4
198.60	2.33	0.46	205	1.6
198.80	2.63	0.52	232	1.8
199.00	2.91	0.57	257	2
199.20	3.16	0.62	279	2.2

Zone 2 Capacity

Groundwater Elevation (ft.)	Velocity (ft/s)	Discharge (cfs)	Discharge (gpm)	Upward Head on Liner (ft.)
196.83	0.00	0.00	0	-0.17
196.90	0.71	0.14	63	-0.1
197.00	1.11	0.22	98	0
197.20	1.64	0.32	145	0.2
197.40	2.04	0.40	180	0.4
197.60	2.37	0.46	209	0.6
197.80	2.66	0.52	234	0.8
198.00	2.92	0.57	257	1
198.20	3.16	0.62	279	1.2
198.40	3.38	0.66	298	1.4

Zone 3 Capacity

Groundwater Elevation (ft.)	Velocity (ft/s)	Discharge (cfs)	Discharge (gpm)	Upward Head on Liner (ft.)
195.34	0.00	0.00	0	0.74
195.40	0.72	0.14	64	0.8
195.60	1.50	0.29	133	1
195.80	2.00	0.39	176	1.2
196.00	2.39	0.47	211	1.4
196.20	2.73	0.54	241	1.6
196.40	3.03	0.60	268	1.8
196.80	3.56	0.70	314	2.2
197.00	3.80	0.75	335	2.4
197.20	4.02	0.79	354	2.6
197.40	4.23	0.83	373	2.8

DOWNSTREAM OVERFLOW/BYPASS (SECTION 3)

PURPOSE

The following calculation determines the capacity of the downstream by-pass pipe.

METHOD

The Bernoulli equation will be used to calculate pipe flow. The equation will be written between a point set at the top of the groundwater level beneath liner (lowest point of liner) and the water surface elevation downstream of the downstream cut off wall. Minor losses will be calculated based on velocity head.

DESCRIPTION OF BY-PASS SYSTEM

The by-pass system operates similar to an artesian aquifer. As the groundwater level rises above the liner elevation, the weight of the gabion mat and the water above the liner causes pressure to build up beneath the liner.. As the groundwater level rises, water will not exit through the by-pass pipe *until the groundwater level beneath the liner exceeds the water surface elevation downstream of the* cutoff wall. An assumption is made that the water surface elevation over the downstream cutoff wall is the same. As the water level rises, the liner will not float because the groundwater level beneath the liner is equal is equal to the water surface elevation on top of it. Once the groundwater level beneath the liner just exceeds the surface water elevation above it, there is the remaining downward head from the weight of the gabion mat to allow water to exit the by-pass.

ASSUMPTIONS

- The liner and gabion acts like a rigid system.
- Velocity head beneath the liner is zero.
- Friction loss in pipe outside the system is negligible.

Saturated Unit Weight of Gabion Mat

Assumptions

Gabion mat in-filled with sand with the following volume distribution

- Gabion stone 70% (S.G. = 2.8)
- Sand 30% with a porosity of 0.4 (S.G. = 2.6)

Therefore:

$$(0.7)(2.8)(62.4\ \text{lb/ft}^3) = 122.3\ \text{lb/ft}^3$$

$$(0.17)(2.6)(62.4\ \text{lb/ft}^3) = 27.6\ \text{lb/ft}^3$$

$$(0.13)(62.4\ \text{lb/ft}^3) = 8.1\ \text{lb/ft}^3$$

$$\gamma_{\text{Gabion}_{sat}} = 158\ \text{lb/ft}^3$$

$$\gamma_{\text{Gabion}_{sat}} = 158\ \text{lb/ft}^3$$

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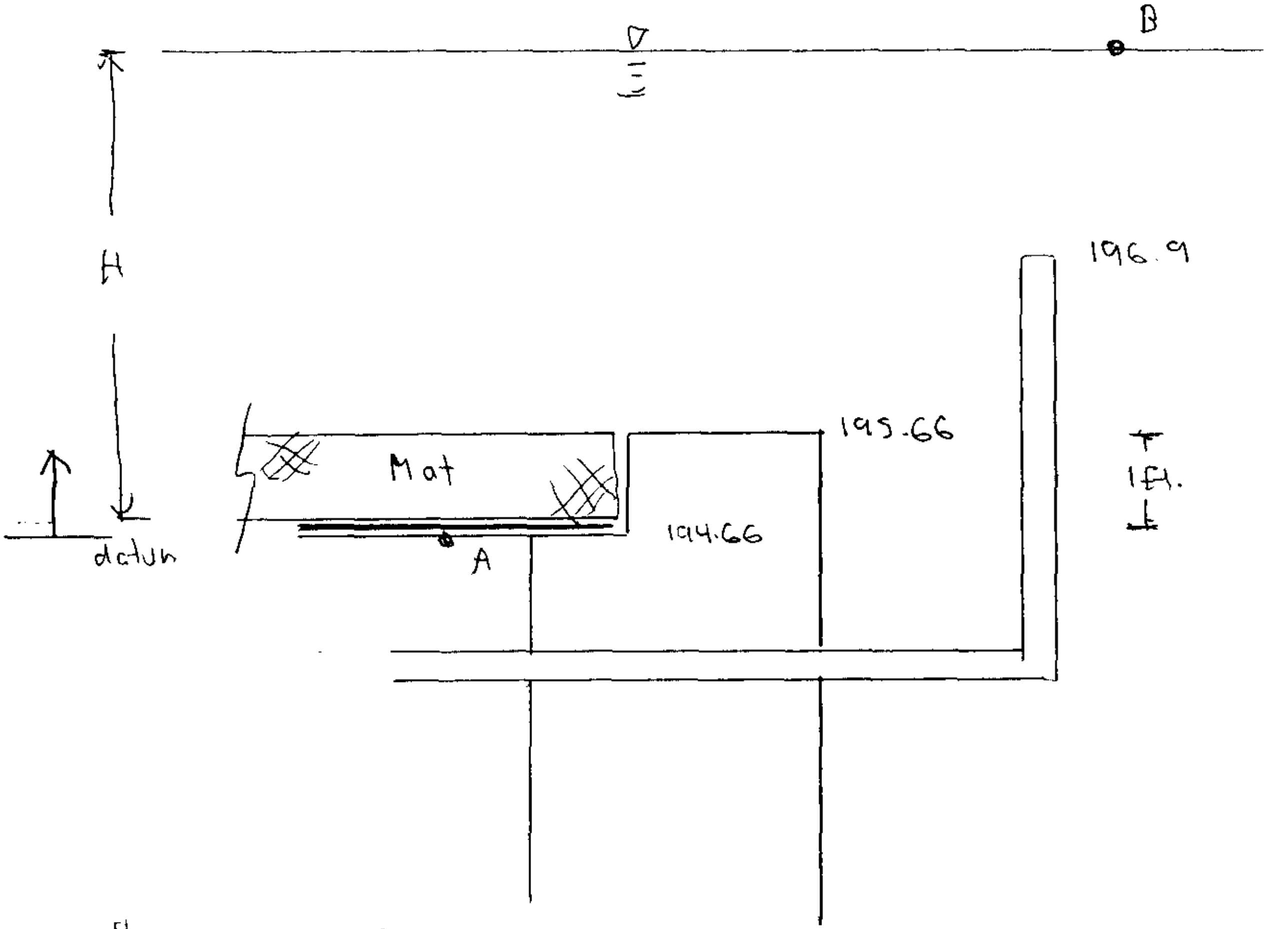
CALCULATION SUMMARY



$$\cancel{Z_A}^{0} + \frac{P_A}{\gamma} + \cancel{\frac{v_A^2}{2g}}^{0} - h_{L_m} = Z_B + \cancel{\frac{P_B}{\gamma}}^{0} + \cancel{\frac{v_B^2}{2g}}^{0}$$

$\frac{P_A}{\gamma}$

$$\frac{P_A}{\gamma} = (H-1) + \left(\frac{1\ \text{ft}\ (\gamma_{Gabion\ sat})}{62.4\ \text{pcf}}\right) \qquad \gamma_{Gabion\ sat} = 158\ \text{pcf}$$

$$\frac{P_A}{\gamma} = H + 1.53$$

SHEET 9 OF 22 PROJECT NO. 95-227-14 PROJECT NAME Spectron
BY ETF DATE 4-11-00 DESCRIPTION
CHK. BY PLS DATE 5-30-00

AR301735

h_{L_m}

- 90° Elbow (1)

  $1.18 \frac{v^2}{2g}$

- Contraction (1)

  $0.25 \frac{v^2}{2g}$

- Butterfly Valve (1)

  $0.15 \frac{v^2}{2g}$ (fully open)

- Backflow Preventer (1)

  $2.5 \frac{v^2}{2g}$ (fully open)

Calculation

$$\frac{P_A}{\gamma} - h_{L_m} = z_B$$

$$H + 1.53 - h_{L_m} = H$$

$$h_{L_m} = 1.53$$

Flow caused from the buildup of pressure beneath the liner can be calculated incrementally to the point of liner float as:

$$h_{L,m} = X$$

where X ranges from 0 to 1.53 ft.

When X = 1.53 ft, the maximum discharge will occur without liner float. At X = 1.54 ft., the liner will begin to float.

Therefore

$$4.08 \frac{V^2}{2g} = X$$

$$V^2 = \frac{2g}{4.08} X$$

$$\underline{V = \left(\frac{2g}{4.08} X \right)^{1/2}}$$

CAPACITY CALCULATIONS
DOWNSTREAM CUT BYPASS PIPE

Incremental Upward Head on Liner (ft.)	Velocity (ft./sec)	Discharge (cfs)	Discharge (gpm)	Comments
0	0.00	0.00	0	Pressure on Liner
0.2	1.78	0.16	70	Pressure on Liner
0.4	2.51	0.22	99	Pressure on Liner
0.6	3.08	0.27	121	Pressure on Liner
0.8	3.55	0.31	139	Pressure on Liner
1	3.97	0.35	156	Pressure on Liner
1.2	4.35	0.38	171	Pressure on Liner
1.4	4.70	0.41	184	Pressure on Liner
1.53	4.91	0.43	193	Pressure on Liner
1.55	4.95	0.43	194	Liner Float

DOWNSTREAM OVERFLOW/BYPASS (SECTIONS 1 and 2)

PURPOSE

The following calculation determines the capacity of the by-pass pipe in Sections 1 and 2.

DESCRIPTION OF BY-PASS SYSTEM

Initially, when the groundwater first begins to rise, the by-pass for Section 1 and for Section 2 undergo weir flow. Once the pipes are full, pressure flow begins. The invert of the Section 2 by-pass is 6 inches below the invert of the by-pass in Section 1. Therefore, both will experience pressure flow at the same time.

METHOD

Both a V-notch and a broad crested weir equation will be used to approximate weir flow for a circular pipe. The Bernoulli equation will be used to calculate pipe flow under the pressure condition. The equation will be written between a point set at the top of the groundwater level beneath liner (lowest point of liner) and the water surface elevation downstream of the downstream cut off wall. The Manning equation will be used to determine friction loss. Minor losses will be calculated based on velocity head.

ASSUMPTIONS

- The liner and gabion acts like a rigid system.
- Velocity head beneath liner is zero.
- Depth of flow in Sections 1 and 2 is 1.5 ft. lower than at the downstream cutoff wall.
- Weir flow in the circular pipes can be approximated as a square broad crested weir and/or a V-notch weir.

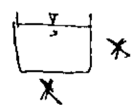
CALCULATION SUMMARY

Weir Flow (Broad Crested)

$$Q = C_w L H^{1.5}$$

Assume equivalent rectangle for 6" ⌀ pipe

$$\pi r^2 = \pi \left(\frac{3}{12}\right)^2 = 0.20 \text{ ft}^2$$



$$x^2 = 0.20 \text{ ft}^2$$

$$x = 0.44 \text{ ft}$$ (Use $x = 0.2'$)

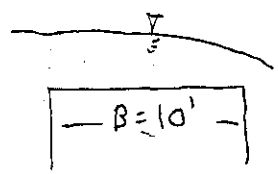
Assume 1/2 pipe flow is only achieved by weir flow

$$Q = C_w L H^{1.5}$$

$L = 0.44$ ft

H = vary from 0 - 0.2

C_w



For

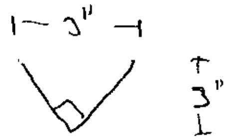
$H = 0.2$; $B = 10'$ $\quad C_w = 2.49$

H (ft)	Q (cfs)	Q (gpm)
0.2	0.10	45

Assume 6" pipe just before flowing full passes 45 gpm via weir flow

Try V-Notch Weir

Assume V-notch for height of radius of pipe



$$Q = C_w H^{2.5}$$

$$Q = 2.54\left(\frac{3}{12}\right)^{2.5}$$

$$Q = 0.08 \text{ cfs}$$

$$Q = 36 \text{ gpm}$$

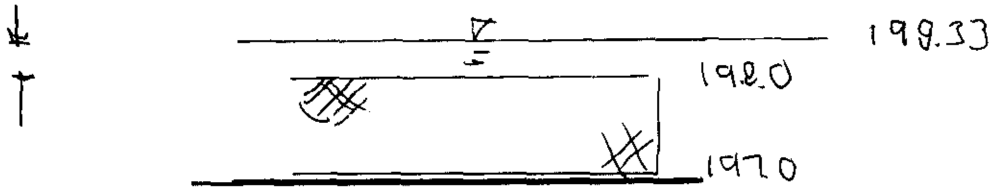
Say 40 gpm max weir flow

Line Condition at Weir Flow

Assume Saturated Flow conditions for worst case

Liner Section (Section 1 and 2)

$r = 4'' = 0.33'$



* Section 1

At full weir flow; groundwater level = 199.7 (199.2 + 0.5)

Downward effective head on liner (h_{eff})

$$h_{eff\,(elev)} = 197.0 + \frac{\gamma_{Gabion\ sat}}{62.4\ pcf} + 0.33$$

$$\gamma_{Gabion\ sat} = 150\ pcf$$

$$h_{eff\,(elev)} = 199.86$$

$$h_{eff} = 199.86 - 199.7 = 0.16\ ft$$

ok
No float

• Section 2

At full weir flow; groundwater level = 199.2 (198.7 + 0.5)

Downward effective head on liner (h_{eff})

$$h_{eff\ elev} = 197.0 + \frac{\gamma_{Gabion\ sat}}{62.4\ pcf} \times 0.33$$

$\gamma_{Gabion\ sat} = 158\ pcf$

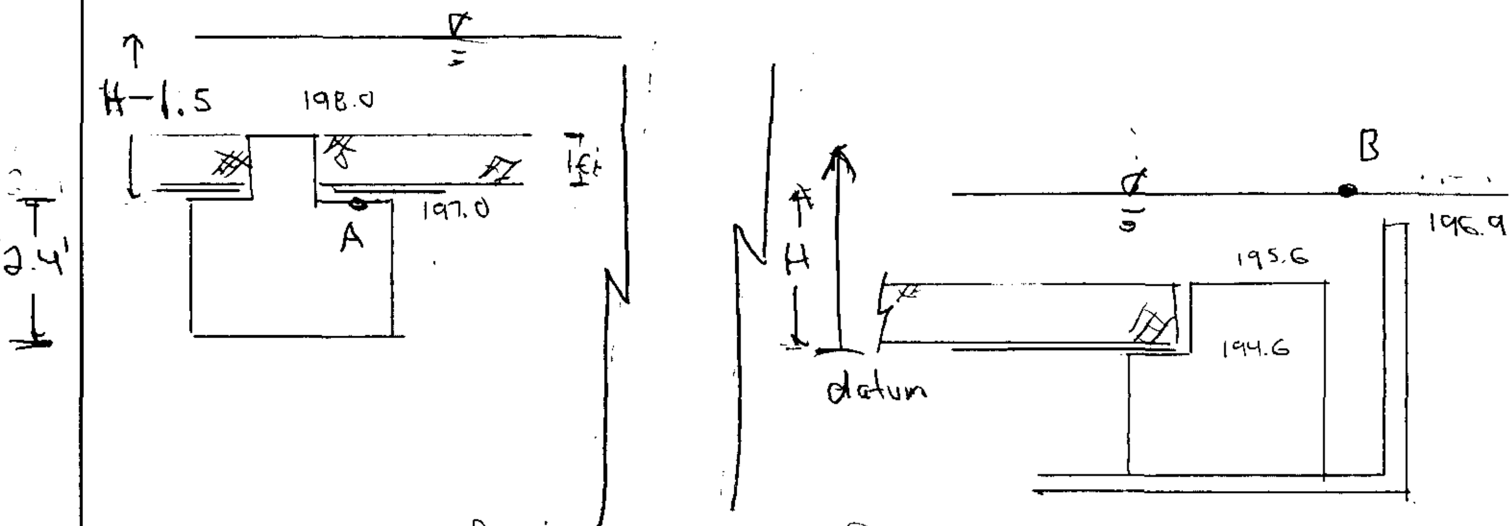
$h_{eff\ elev} = 199.86$

$h_{eff} = 199.86 - 199.2 = 0.66$ ft. OK

No float

Pressure Flow

Section 1 and Section 2 overflow have the same liner grade at overflow.



$$z_A + \frac{P_A}{\gamma} + \cancel{\frac{v_A^2}{2g}}^{0} - h_L = z_B + \cancel{\frac{P_B}{\gamma}}^{0} + \cancel{\frac{v_B^2}{2g}}^{0}$$

$$z_A + \frac{P_A}{\gamma} - h_L = z_B$$

$$\frac{P_A}{\gamma}$$

$$\frac{P_A}{\gamma} = (H - 1.5 - 1.0) + \left(\frac{1 ft (\gamma_{Gabion\ sat})}{62.4\ pcf} \right) \qquad \gamma_{Gabion_{sat}} = 158\ pcf$$

$$\frac{P_A}{\gamma} = H + 0.032$$

h_L

- Friction Loss

$$h_f = 2.61 \times 10^{-4}\, L V^2$$

- Minor Losses
  - 90^0 Elbow

    $$1.18 \frac{V^2}{2g}$$

  - Contraction

    $$0.25 \frac{V^2}{2g}$$

  - Butterfly valve

    $$0.15 \frac{V^2}{2g} \quad \text{(fully open)}$$

  - Backflow preventer

    $$2.5 \frac{V^2}{2g} \quad \text{(fully open)}$$

Equation

$$Z_A - Z_B + \frac{P_A}{\gamma} = h_L$$

$$h_{L_f} + h_{L_{minor}} = 2.4 - H + H + 0.032$$

$$h_{L_f} + h_{L_m} = 2.43$$

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"Engineering for the Environment"™

Flow caused from the buildup of pressure beneath the liner can be calculated incrementally to the point of liner float as:

$$h_{L_{ed}}\ h_{L_{m}} = 2.4 + X$$

where X ranges from 0 to 0.032 ft

When X = 0.032 ft., the maximum discharge will occur without liner float. At X = 0.033 ft., the liner will begin to float.

This is such a small increment, therefore, the maximum discharge capacity without liner float will be calculated

SHEET 20 OF 22 PROJECT NO. 95-227-84 PROJECT NAME Spectrum
BY rhz DATE 4-11-98 DESCRIPTION
CHK. BY PLS DATE 5-30-00

AR301746

Sectional OverFlow

- Data

90° Elbow = 8

Contraction = 1

Butterfly Valve = 1

Backflow Preventer = 1

L = 630

- Calculation

$$(2.61 \times 10^{-4})(630)V^2 + 12.34\frac{V^2}{2g} = 2.43$$

$$0.36\,V^2 = 2.43$$

$$V = 2.60\ ft/sec$$

$$Q = (2.60\tfrac{ft}{sec})(\pi)(\tfrac{2}{12}ft)^2 = 0.23\ cfs$$

$$Q = 102\ gpm$$

Section 2 Overflow

- Data

  90° Elbows = 4

  Contraction = 1

  Butterfly Valve = 1

  Backflow Preventer = 1

  L = 322

- Calculation

$$(2.61 \times 10^{-4})(322)\,U^2 + 7.62\,\frac{U^2}{2g} = 2.43$$

$$0.20\,U^2 = 2.43$$

$$U = 3.47 \text{ ft/sec}$$

$$Q = (3.47 \text{ ft/sec})(\pi)\left(\frac{3}{12}\text{ ft}\right)^2 = 0.30 \text{ cfs}$$

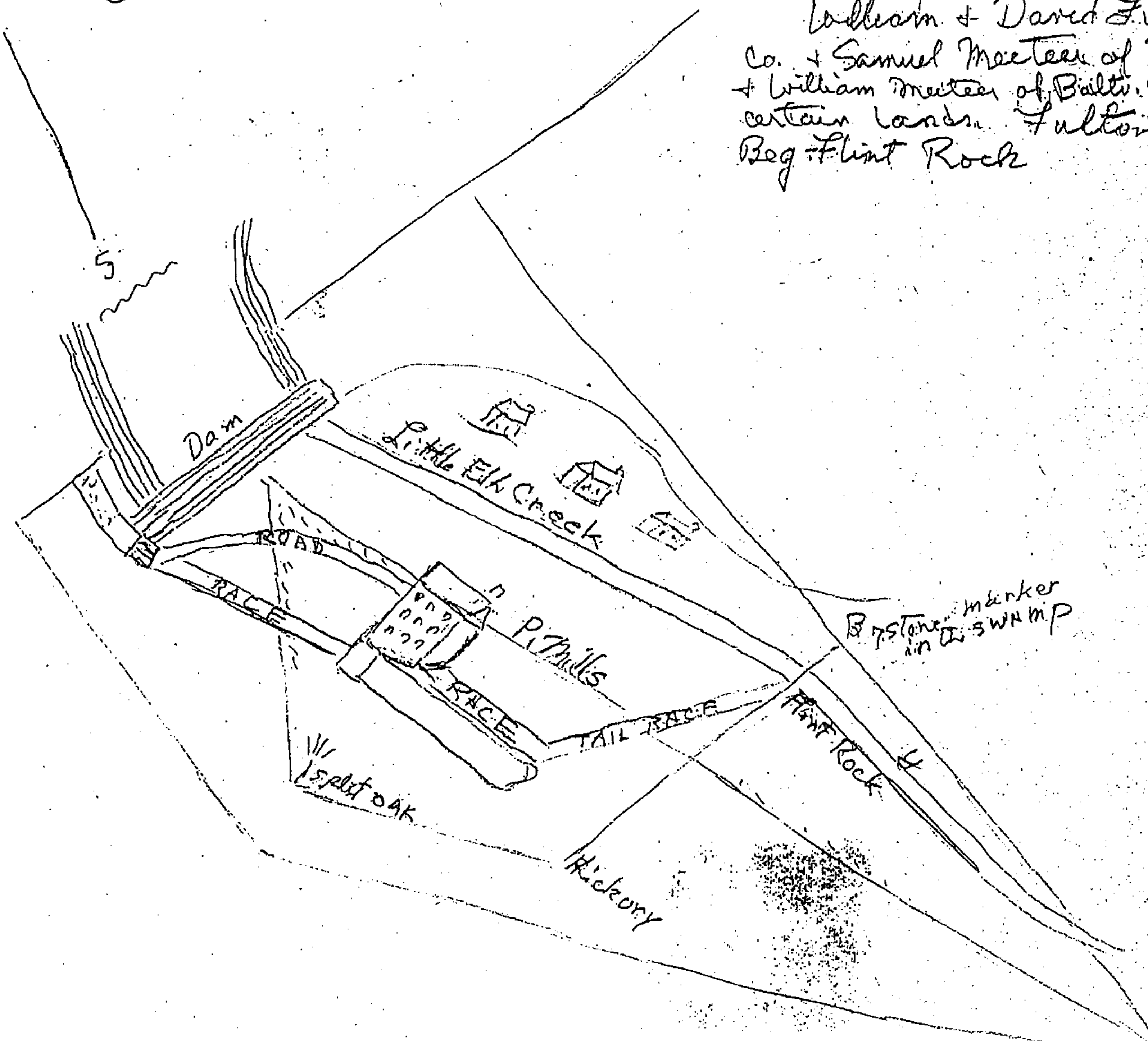
$$Q = 136 \text{ gpm}$$

ATTACHMENT 5

HISTORIC SITE MAPS AND PHOTOS

William ... lton

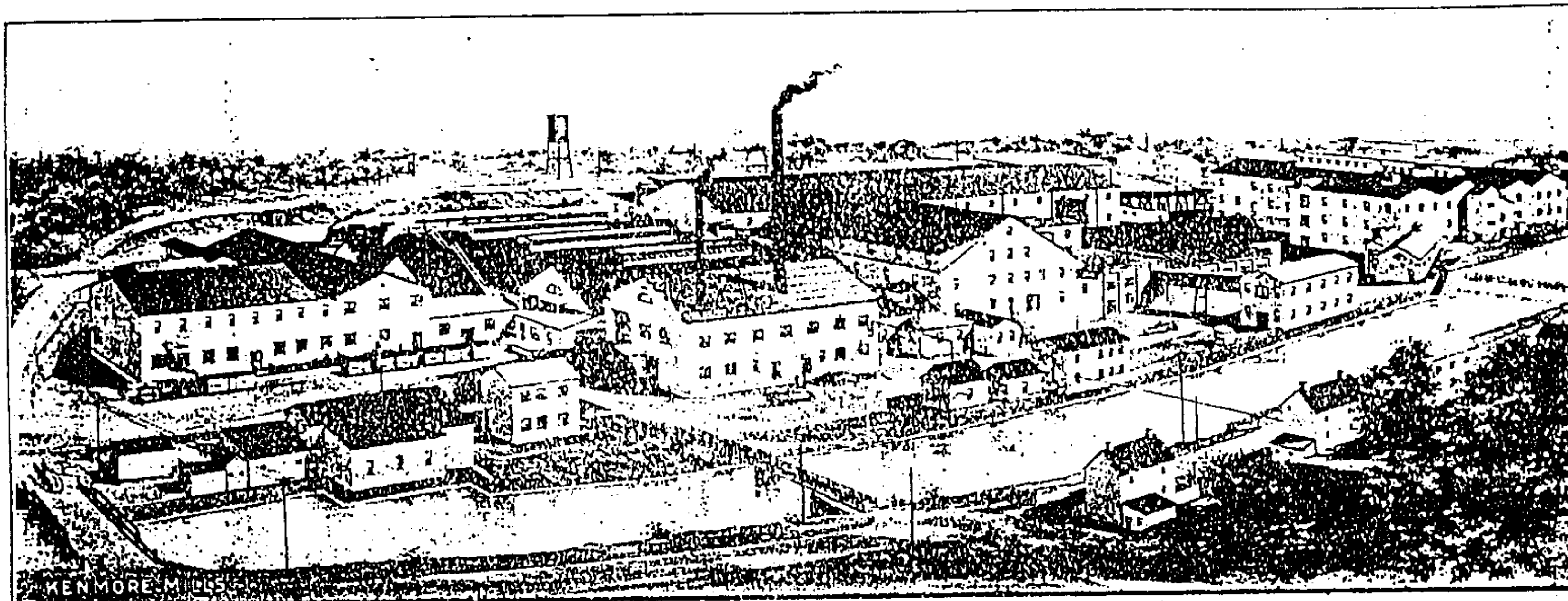
July 1846
William + David Fulton of Cecil Co. + Samuel Meeteer of Ne Castle Co, De + William Meeteer of Balti. Agree to exchang certain lands. Fulton's to Meeteers
Beg Flint Rock

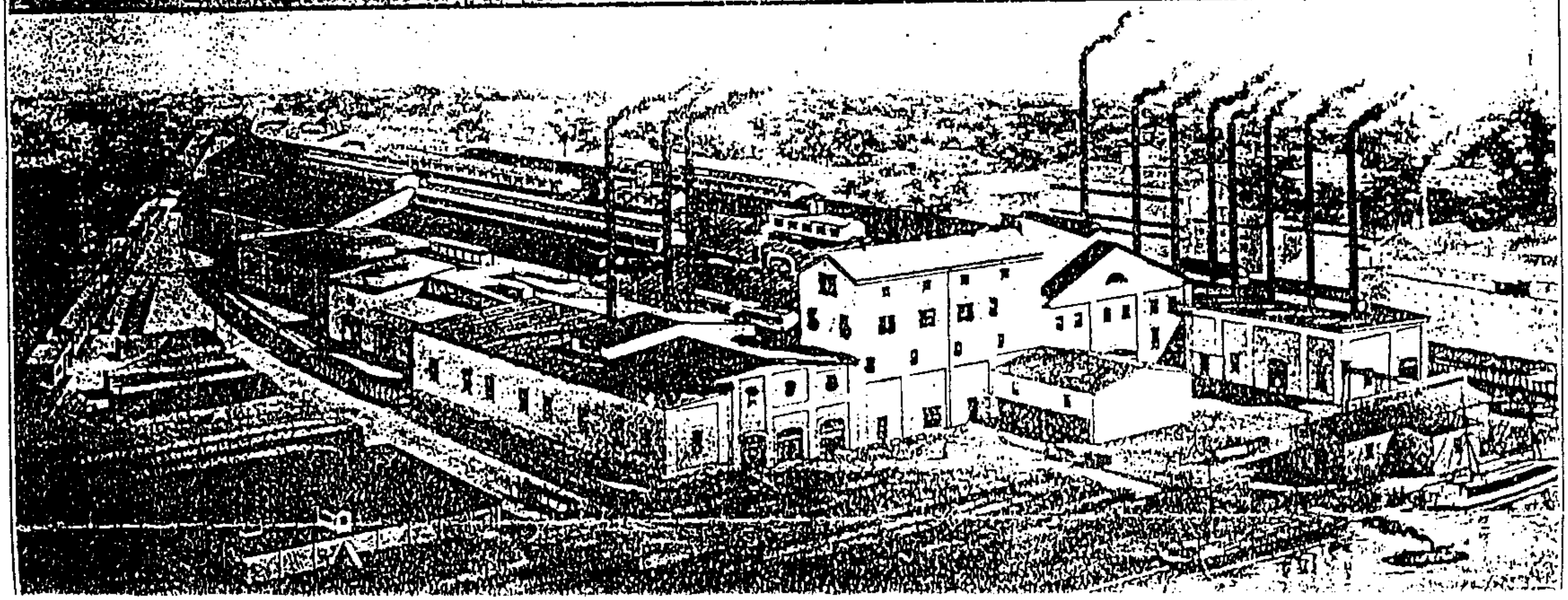


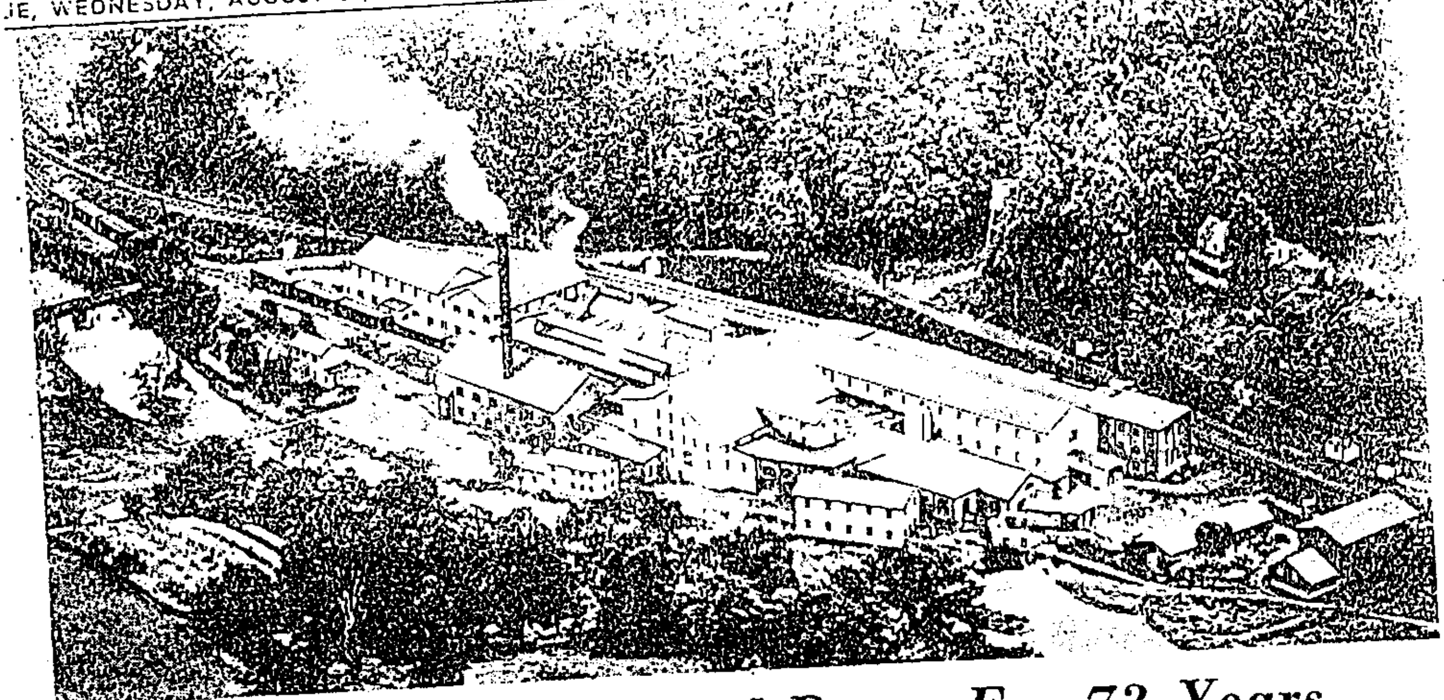
Harold and Anne Copley
1610 Yeatmans Mill Rd.
Newark, DE 19711
(302) 234-0341

KENMORE MILLS and RADNOR MILLS

Important Cecil County Industries







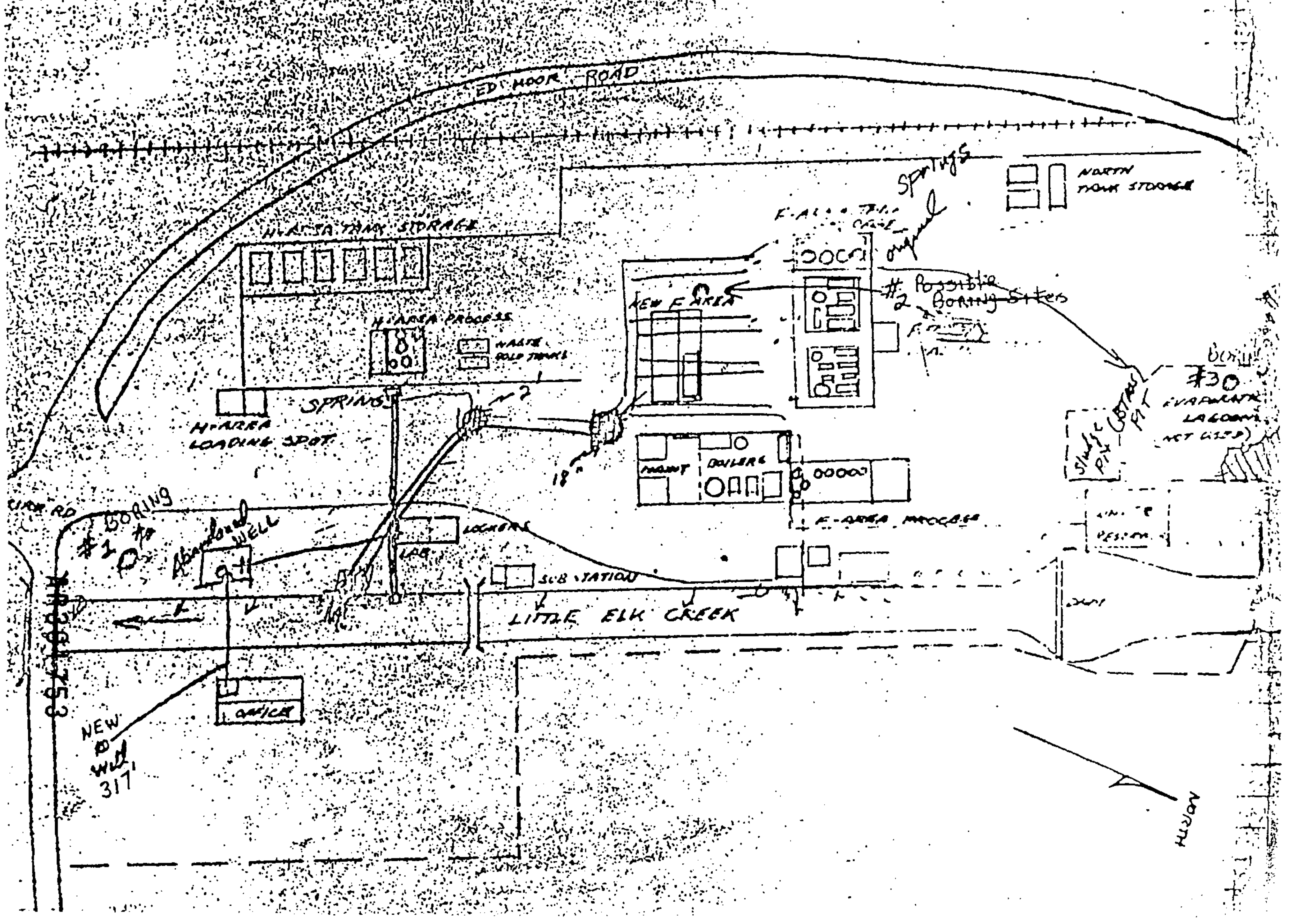
Kenmore Mills Produced Paper For 73 Years

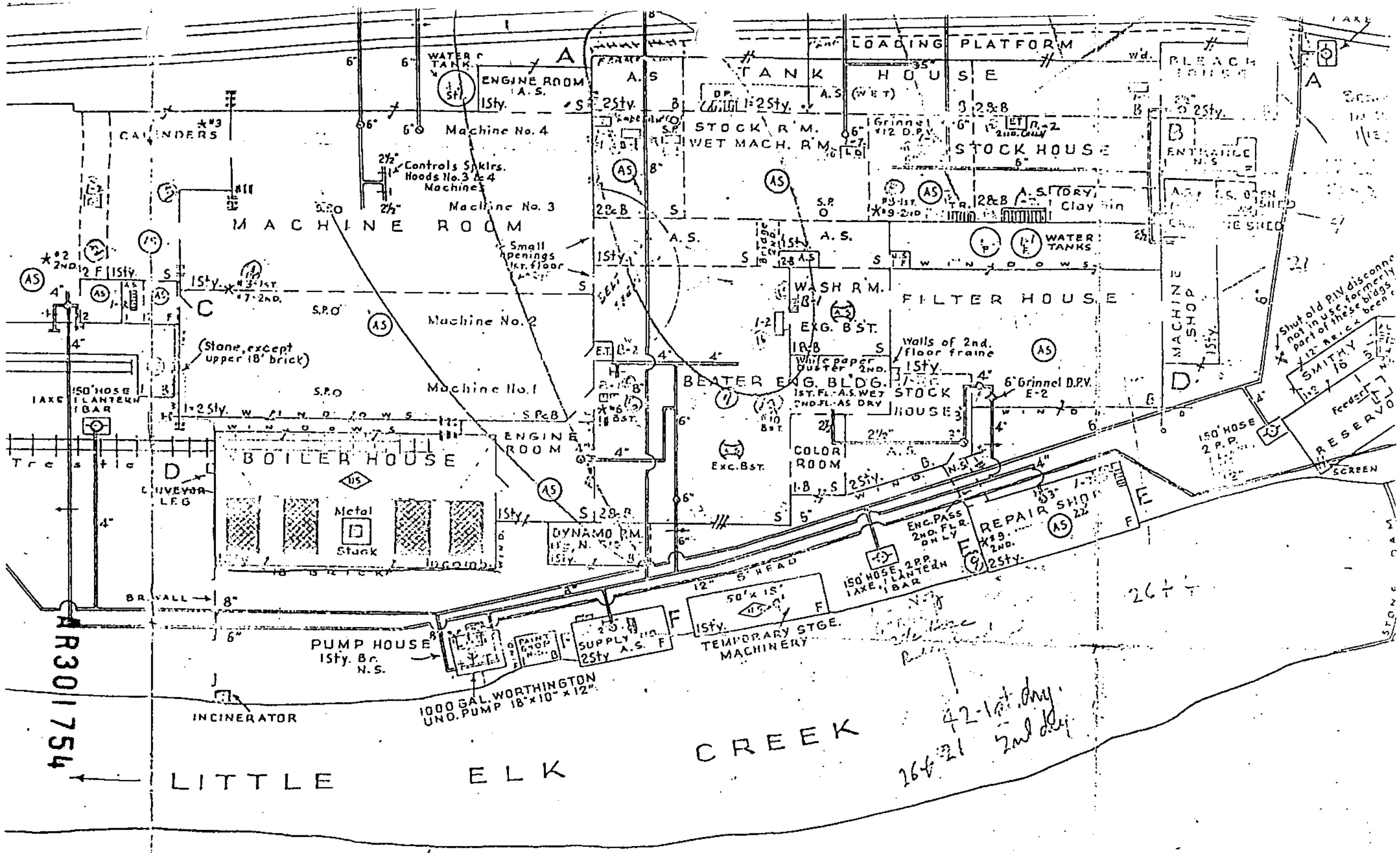
The sketch above shows Kenmore Mills, Providence, which was built in 1881 and ceased operation when it was destroyed by fire in 1954. The 1919 Historical and Industrial edition of the Cecil Whig said: Kenmore Mills, the capacity of this mill-is about 85,000 pounds of paper daily, comprising book paper, machine finish and super calendar. The plant employs hands or more, on six-day weekly runs, under the ploys 200 hands or more, on six-day weekly runs, under the management of Superintendent David Lindsay. This has made Providence, where the plant is located, the largest and most thrifty village in the Fourth district.

DIAGRAM 5

NO SCALE

REV'D 2/23/73, 8/1/74





AR301754

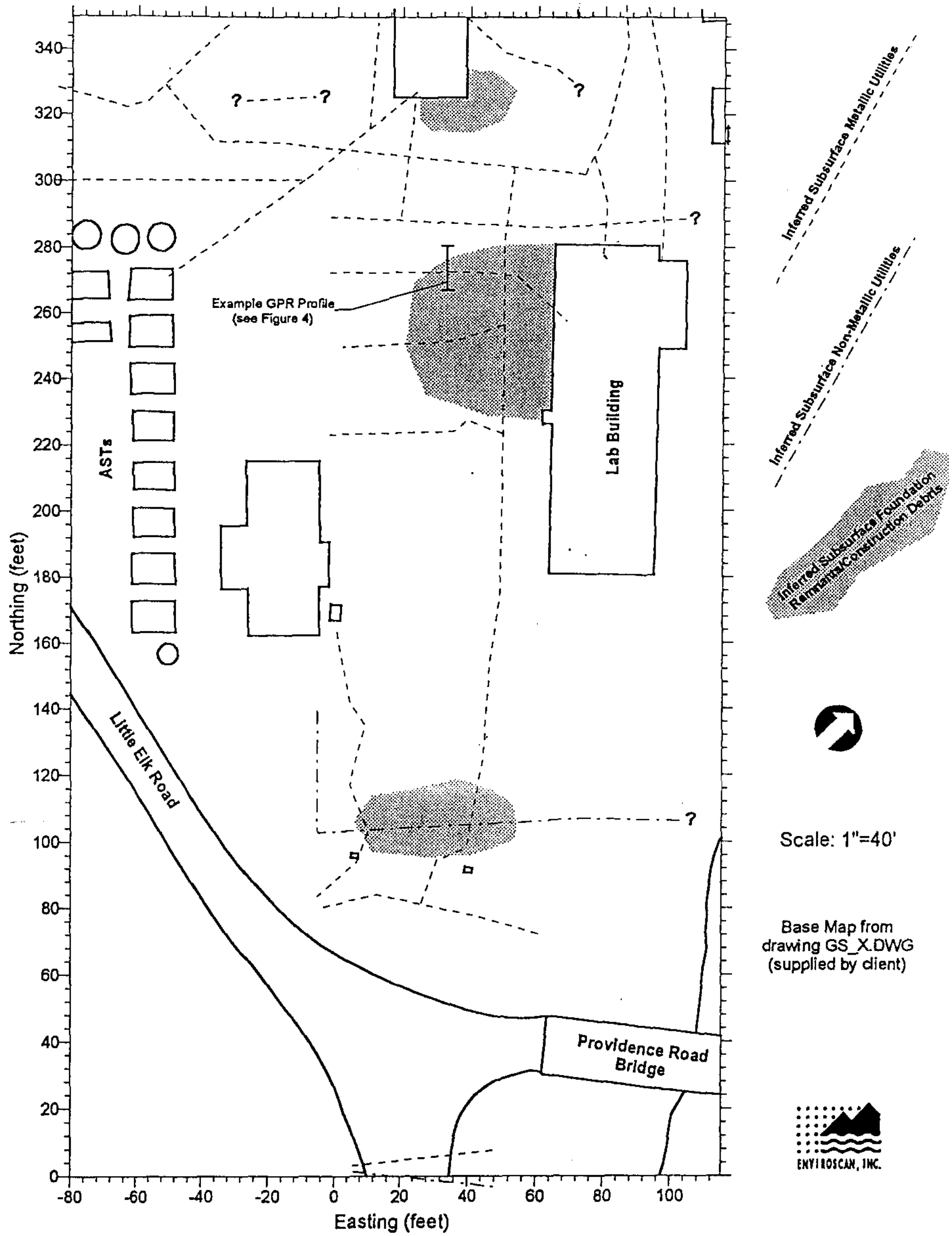
- Kenmore Mills -

THE JESSUP & MOORE PAPER COMPANY

- Providence, Md. -

Dec. 1947 ·WFS· Scale 1in.=50'

JAN 31 1958

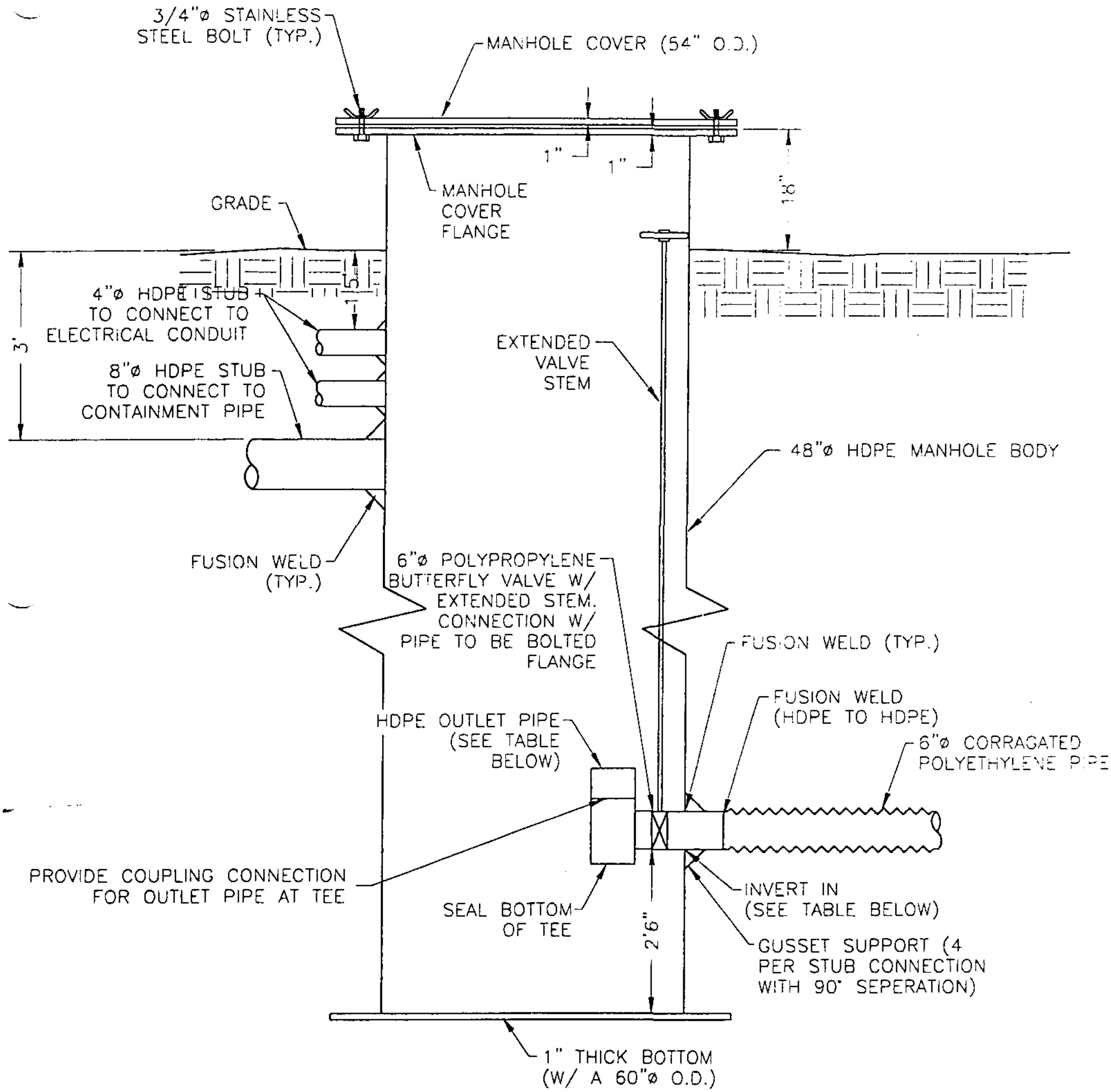


AR301755

Figure 3 EM/GPR Survey Interpretation

Galaxy/Spectron Site
Little Elk and Providence Roads
Elkton, MD

Enviroscan, Inc.
Project Number 109513
Rev. 06/30/96



TYPICAL COLLECTION SYSTEM SUMP (MANHOLE)

1"=2'

AR301756